CIVIL ENGINEERING

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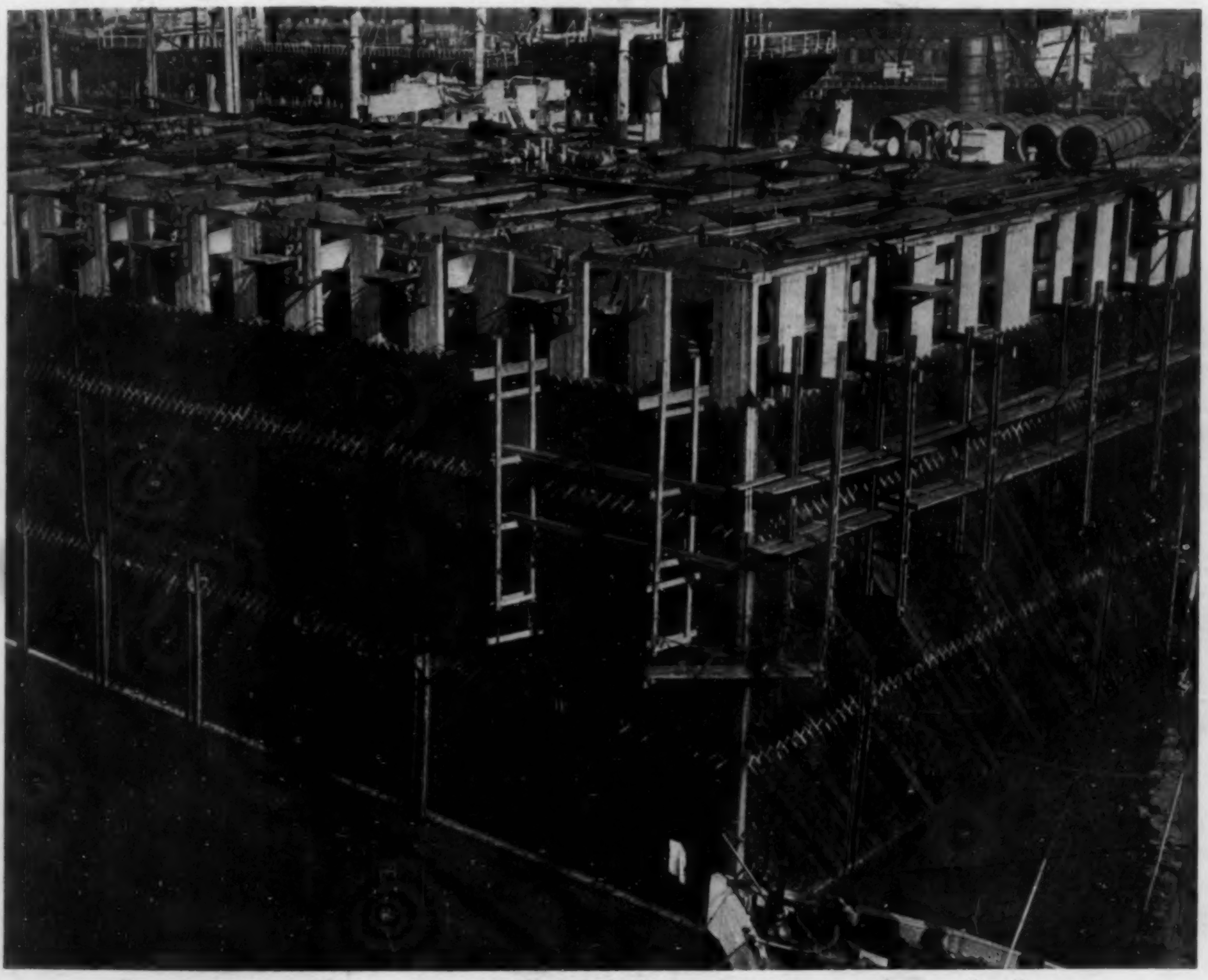

Caisson for Center Anchorage of Twin Suspension Spans of San Francisco–Oakland Bay Bridge
Has Been Floated to Location and Will Rest on Rock, Which Is from 187 to 210 Ft Below the Water Surface

Volume 4 AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852 *Number 4*

APRIL 1934

Among Our Writers

C. H. Purcell was the first bridge engineer for the Oregon State Highway Department. In 1920 he was appointed district engineer of the U. S. Bureau of Public Roads. In 1928 he was appointed state highway engineer for the California Division of Highways. In October 1929, when the San Francisco-Oakland Bay Bridge Commission was appointed, he was made a member and secretary of this commission. The commission adopted the bridge plan prepared under his direction.

J. W. Orton's professional experience consists of 9 years of diversified civil engineering training with two consulting engineering firms of Kansas City, Mo.—Burns and McDonnell, and Black and Veatch—followed by 7 years as assistant civil engineer in charge of filtration structures construction for the Detroit Water Board.

Donald P. Barnes returned from his studies as a Freeman Traveling Scholar of the Society in September 1933. At the present time he has charge of experimental investigations on certain phases of the Colorado River Aqueduct for the Metropolitan Water District of Southern California.

Raymond P. Pennoyer, after graduation from Stevens Institute of Technology, entered the employ of the Carnegie Steel Company, where he has remained ever since except for two years with the A.E.F. This included service with the 15th Engineers and with the Tank Corps. He is a Lieutenant Colonel in the Infantry Reserves.

A. C. Barrow gained the first 15 years of his experience successively as instrumentman, surveyor, engineer in charge of railroad location and construction, and as consulting engineer, in central Kentucky. For the past 7 years he has been teaching at the Alabama Polytechnic Institute.

Leon Blog, after graduating from Cornell University, was industrial engineer with the Goodyear Tire and Rubber Company for 5 years. For 11 years he has served the City of Los Angeles, the last 10 years in the Division of Bridges and Structures.

R. D. Goodrich was assistant and deputy State Engineer of Wyoming 1906–1909. After two years of practice in irrigation engineering he became engineer for the cities of Cheyenne, Ypsilanti, and Lansing. After eight years on river and flood control work in China, in 1927 he became Associate Professor of Civil Engineering at the University of Wyoming. He is the author of numerous published articles on technical subjects.

Daniel T. Webster was formerly with the architectural firm of McKim, Mead and White. He is a vice-president of the New York Building Congress. As a member of the National Panel of Arbitrators of the American Arbitration Association, he has served as arbitrator in some of the most important matters submitted to the association for arbitration.

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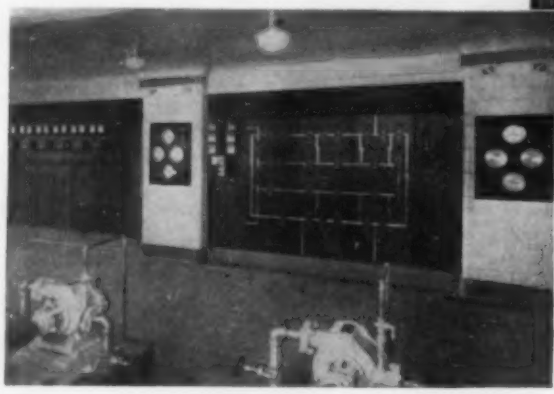

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CIVIL ENGINEERING

APRIL 1934

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NUMBER 4

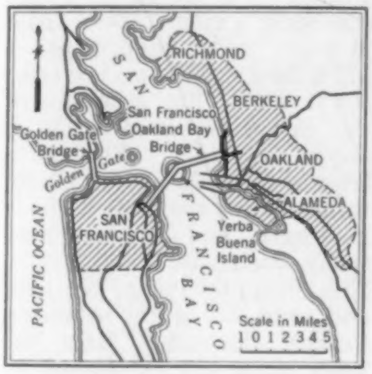
San Francisco–Oakland Bay Bridge

Construction of Huge Public Works Project Is Progressing Rapidly

By C. H. PURCELL

ASSOCIATE MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
CHIEF ENGINEER, SAN FRANCISCO–OAKLAND BAY BRIDGE; AND CALIFORNIA STATE HIGHWAY ENGINEER, SAN FRANCISCO, CALIF.

ONE of the largest bridge projects ever undertaken by man is the $77,000,000 structure now under construction to connect San Francisco with its sister cities across San Francisco Bay. Its length, its height above water, the depth of its piers, and the length of its twin suspension spans across the two miles of water between San Francisco and Yerba Buena Island, are all remarkable. The work, financed by the aid of funds secured through the Reconstruction Finance Corporation, was begun in July 1933. Already the construction of the foundations and anchorages is well advanced. Among the outstanding features of this project is the use of a new type of compressed air flotation caisson. The caisson for the center anchorage of the twin suspension spans in the west-channel is the largest yet built. One of the tower piers, extending 218 ft to rock through 160 ft of mud, has established a record for depth. These and other features of this stupendous job are here described by Mr. Purcell. He also outlines the reasoning which led to the selection of the route and the various types of bridge construction used.



AN area approximately six miles square, at the tip end of a peninsula, is occupied by the City and County of San Francisco. Its population is 634,394 and it is a financial, business, industrial, and shipping center. Its bay shore is lined solidly with docks for ocean-going ships. Electric and cable street cars carry all the downtown passenger traffic, although there are buses in a few outlying sections.

Oakland, on the mainland across the bay from San Francisco, is the principal city and county seat of Alameda County and has a population of 284,063. Welded into Oakland within Alameda County are the cities of Alameda, with a population of 35,033; Berkeley, with one of 82,109; and smaller communities. Oakland, a fast-growing industrial center, has shipyards and excellent rail and water terminal facilities on its bay shore.

The residential advantages of Oakland, Berkeley, and Alameda, together with the large payroll of San Francisco, have developed a class of 50,000 commuters who live in Alameda County, go to San Francisco to work each morning, and return each evening. These commuters purchase 35,000,000 trips across San Francisco Bay each year on ferries operated by two companies, which ferries are part of two Alameda County street railway lines. In addition to this, automobile ferries annually carry 5,000,000 automobiles with 10,000,000 passengers between San Francisco and Alameda counties. The ferry boats operate on a 20-min schedule and require approximately 20 min to cross in clear weather.

INVESTIGATIONS PROVE BRIDGE FEASIBLE

As early as 1856, a bridge between San Francisco and Alameda County was discussed. However, no serious movement toward substituting a bridge for ferries got under way until 1921, when public-spirited citizens raised funds for preliminary surveys, one of which was made by Ralph Modjeski and J. Vipond Davies, both Members Am. Soc. C.E. These engineers reported that it was possible to bridge San Francisco Bay and dispelled the old superstition that there is no bottom to the bay.

Private corporations and promoters, totaling 35, petitioned the San Francisco Board of Supervisors for a franchise to construct this bridge. San Francisco, being construed to be on the left bank, was deemed to have the right to grant a franchise rather than Alameda County. So competitive were the applicants for a franchise to build the San Francisco–Oakland Bay Bridge that they destroyed one another's opportunities, and none was able to show sufficient strength to win a franchise.

Citizens strongly in favor of eliminating the antiquated ferry system caused the State of California to investigate the possibility of a state-owned bridge. In 1929 a commission was appointed jointly by President Hoover and Governor Young to investigate the economic necessity and engineering feasibility of a bridge across San Francisco Bay and to make recommendations. This commission reported that there was an economic necessity for such a bridge and that it was feasible, and proposed a design which, with alterations, is now being built under the California Toll Bridge Authority.

BIRD'S-EYE VIEW OF SAN FRANCISCO–OAKLAND BAY BRIDGE
A California Toll Bridge Authority Project Begun in July 1933
Construction on Anchorages and Piers Now Well Advanced
Estimated Construction Period, Four Years

Construction was placed under the supervision of the San Francisco–Oakland Bay Bridge Division of the State Department of Public Works. On December 15, 1932, the Reconstruction Finance Corporation signed a contract to purchase from the California Toll Bridge Authority the bonds for the construction of the bridge proper, and the Toll Bridge Authority agreed to build and finance the approaches out of state funds and to maintain the entire structure with state highway funds on its completion. Construction was started on Yerba Buena Island, July 9, 1933, with appropriate ceremonies. On that occasion President Roosevelt set off by telegraph a blast which broke ground for the island section of the work.

The San Francisco–Oakland Bay Bridge will be a double-decked vehicular structure with roadways 58 ft wide (Fig. 1), having a total length, including approaches, of $8^{1}/_{4}$ miles. From Rincon Hill in San Francisco it will extend in a northeasterly direction to Yerba Buena Island, bridging an expanse of 10,450 ft with twin suspension spans.

Yerba Buena Island, in San Francisco Bay, midway between Oakland and San Francisco, is an irregular shaped outcropping of sandstone, about 3,000 ft across at its widest point, and rises 340 ft above the surface of the water. It is occupied jointly by U.S. Army, Navy, and Lighthouse services.

A main approach on the line of the bridge and two curved ramps, one for ingress and another for egress, will connect with the bridge at Rincon Hill.

The interurban cars on the lower deck will come off the bridge to a terminal as yet unlocated. The truck roadway of the lower deck will leave the bridge on a separate incline, so that the San Francisco approach will provide four separate automobile ramps in addition to an elevated electric train structure.

A SUSPENSION BRIDGE TOWER AS IT WILL APPEAR AGAINST THE SAN FRANCISCO SKY LINE

The West Bay is crossed by twin suspension bridges of equal length, each with a main span of 2,310 ft and side spans of 1,160 ft. The clearance of these suspension bridges ranges from 180 to 214 ft above the water. The cable of the western suspension bridge is anchored on Rincon Hill in a concrete monolith, and in the middle of the channel in a concrete center anchorage, fastened to rock, of sufficient weight to resist the live and dead-load pull of the cables. The eastern suspension bridge likewise is anchored in the middle of the channel in this center pier and, on its easterly end, in the rock of Yerba Buena Island, where the cables will extend into tunnels bored in the rock, to eye-bars and grillages set in concrete. The island crossing will be accomplished by means of a double-decked vehicular tunnel with a bore 51 ft high, $65^{1}/_{3}$ ft wide, and 540 ft long. It is to be lined with steel ribs and plates imbedded in concrete.

From the east portal of the tunnel to the east shore of the island the bridge will be carried on a concrete and steel elevated structure for 2,500 ft, curving slightly to the right. From the shore of the island it follows a line almost due east toward Alameda County. The span over the shipping channel, of the cantilever type, is 1,400 ft long, and is only exceeded in length by the Quebec Bridge and the Firth of Forth Bridge. The cantilever bridge consists of a main 1,400-ft span, in which the suspended span is 576 ft long and the anchor arms 508 ft each. The clearance of the cantilever span is 185 ft above the water. The cantilever is followed by five simple through-truss spans, each 504 ft in length, and 14 deck-truss spans, each 288 ft in length. This brings the bridge to a rock and sand fill in the Alameda County tidelands. Here the lower deck will be forked to permit the upper deck to come down between the forks at grade. A toll house and administration building will be located on this fill.

At the West Bay crossing the substructure is made up of eight piers, three of which (Piers A, B, and W-1) are on the San Francisco shore, where filled land requires submarine construction methods. Off the shore line of San Francisco the first pier now completed, W-2, has a depth ranging from 79 to 100 ft, and was constructed inside of single-wall steel sheet piling by the open-coffer-dam method. Piers W-3, W-4, W-5, and W-6 range in depth from 105 to 218 ft and, like W-2, support the towers of the twin suspension bridges, except for W-4, which forms the substructure of the concrete center anchorage

between the twin suspension bridges. These are being constructed by dredging in open wells, with steel and timber caissons floated by compressed air entrapped within the steel dredging wells, which are domed during the period of flotation.

In the East Bay substructure, Piers YB-1, YB-2, YB-3, and E-1 are all concrete columns built on land to support a steel viaduct. Pier E-2 will be constructed by the steel-sheet-piling, open-cofferdam method, whereas Pier E-3, extending to an elevation of −225 ft, and Piers E-4 and E-5, each extending to an elevation of −175 ft, are being built in a false-bottom caisson by the open-dredging well method. All the other piers in the shallow water to the east are of reinforced concrete construction carried down to an elevation of −45 or −50 ft. They are carried on untreated Douglas fir piles averaging about 85 ft in length. A single-wall cofferdam of steel sheet piling is being used for this construction.

BACKGROUND OF DESIGN AND LOCATION

In choosing the location of the bridge, two factors were paramount. First, borings disclosed a high ridge of rock between San Francisco and Yerba Buena Island along the line now followed by the bridge. On either side of this ridge the rock slopes to depths which would have greatly increased the cost of foundations. Second, the provision of facilities for traffic distribution required that the end of the structure should be as close as possible to centers of traffic and yet far enough away from them to avoid congestion.

A number of factors influenced the design. Rock for the anchorage of the suspension cables was not available at the west end on Rincon Hill, in San Francisco; therefore a concrete monolith was built for the purpose. At the end of the twin suspension bridges on Yerba Buena Island, ample rock was found, and therefore the cables were there anchored in rock tunnels. A vehicular tunnel, rather than a cut, was designed to carry traffic across the island because a steep cut would have created the hazard of dangerous slides. Also, whatever slight saving might have been realized by a cut would have been offset by the cost of building bridges over it to accommodate the U. S. Army, Navy, and Lighthouse personnel stationed on the island.

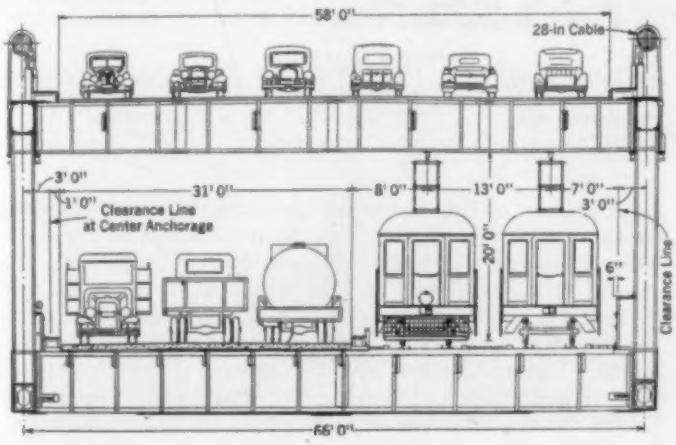


FIG. 1. CROSS SECTION OF SAN FRANCISCO–OAKLAND BAY BRIDGE

For the East Bay a cantilever structure was chosen because suitable foundation material for a cable anchorage is lacking on the Alameda shore, where rock is not available above an elevation of −300 ft. For the remainder of the bridge, simple through trusses, deck-truss spans, and approach fill in combination were chosen for their economy.

Before the final decision was made as to type of structure and major dimensions, a number of alternative designs were studied, particularly for the West Bay crossing, for which the first structure considered was a cantilever with four 1,700-ft spans. This was followed by a number of proposals involving a suspension bridge. The idea of placing two such bridges of the usual type end to end, with a common anchorage between them, was among the first to receive consideration. It required spans of nominal length and involved no principles of design not already thoroughly proved. It is of interest that this type was finally adopted.

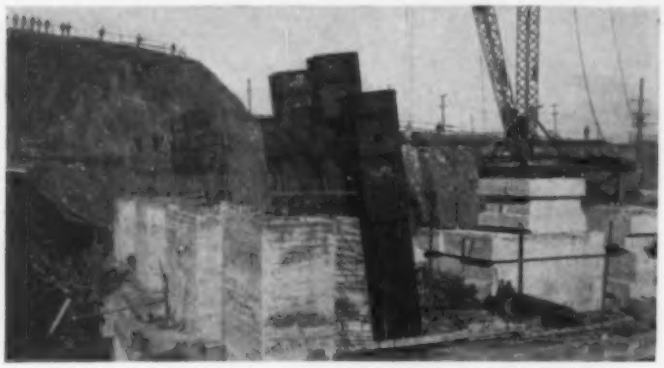
THE PROPOSED CANTILEVER SPAN AGAINST THE OAKLAND SHORE

Serious consideration was given to a suspension bridge consisting of three main spans, each 2,380 ft long. Preliminary designs were prepared for such a structure both with and without auxiliary tie cables. Another plan studied for this crossing was a single 4,100-ft span with 2,000-ft side spans. Many other variations were worked out and estimated, and several months were spent in weighing their relative merits before a final decision was made. Channel clearances, stiffness, cost, and many other factors entered into the final decision to build twin suspension spans.

Pier W-2, at the San Francisco pierhead line, presents nothing unusual in its construction. A single-wall cofferdam of steel sheet piling was driven around the site and internally braced by heavy timber framing. The mud within it was removed by buckets and the rock surface thoroughly cleaned. Under-water concrete was placed by means of $3^1/_2$-cu yd bottom-dump buckets to within about 15 ft of the water surface. After this concrete had hardened, the water within the cofferdam was pumped out and the remaining concrete was placed in the dry.

Piers W-3, W-4, W-5, and W-6 are now being constructed in water as much as 105 ft deep, and must be carried down from 40 to 150 ft further through mud. A tidal current of $6^1/_2$ knots an hour was assumed in the design. In addition, it was necessary to anticipate rather rough water during storms. These factors led to the adoption of a floating caisson of wood and steel construction incorporating several unique features. Since these four piers are all similar, the method of construction of only one, W-4, which is the largest, will be described in detail.

In plan, Pier W-4 is 92 ft by 197 ft. The caisson has a structural steel cutting edge $17^1/_2$ ft deep, and its bottom is pierced by 55 holes 15 ft square. Immediately above the cutting edge the square shafts or wells are transformed to circular form of the same diameter and extend vertically as steel cylinders. The outside wall of the caisson is built up of heavy timber, the timber walls and steel cylinders being all tied together by steel bracing. Practically all the steel work is electric-arc welded. Flotation of this caisson is accomplished by an ingenious method suggested by Daniel E. Moran, M. Am. Soc. C.E., and employed with modifications for the first time on the San Francisco–Oakland Bay Bridge. In



SAN FRANCISCO CABLE ANCHORAGE WITH ANCHOR GIRDERS IN PLACE

place of the usual method of installing false bottoms in the wells, semispherical steel domes are welded on top of open-bottom wells. This traps air in the cylinders and provides some buoyancy. Further buoyancy is supplied by pumping additional air into these chambers.

When towed to the site, the caisson had been built up to a total height of $77^1/_2$ ft and was drawing $20^1/_2$ ft of water. It was anchored in place by 24 concrete T-shaped anchors, each capable of holding up to 125 tons. Concrete was then poured into the spaces between the steel cylinders and the outside wall until the caisson floated with about 15 ft of freeboard. It was then necessary to extend the height of the cylinders and outside wall. The operation of building up the cylinders involved several successive steps. First, the air within a cylinder was released. The dome was then burned off and one or two 20-ft extensions of cylinder were welded in place. The dome was next welded on the top of the extension; and finally the air pressure was raised to the desired amount. Obviously, only a few domes could be removed at a time or too much of the buoyancy would be destroyed. The process of alternately building up the caisson and sinking it deeper in the water with concrete was continued until the cutting edge rested in the mud. The domes were then finally burned off, and the sinking was continued as for an ordinary open-well caisson.

After the caisson is landed on solid rock, a 30-ft concrete seal will be placed. Above this seal the wells will not be filled with concrete, except for three in each corner of Pier W-4. The pier itself will be capped with a heavy concrete slab, above which the superstructure will be erected.

Piers W-2, W-3, W-5, and W-6 will carry steel towers supporting the suspension bridge. Pier W-4, however, will form the base for a great rigid concrete structure reaching 294 ft above the water surface. It will be the common anchorage for the cables of the two suspension bridges extending east and west from it. Within the top of this central anchorage all four cables will be terminated. By means of the usual strand shoes and eyebars, the south cable of the west span will be anchored to one side of a heavy built-up steel plate, to the other side of which the south cable of the east span will be anchored. This plate, in turn, will be rigidly tied into the concrete by means of a structural steel A-frame. In this manner any inequality of load between the two south cables will be transmitted to the anchorage. The north cables will be similarly treated. This anchorage has been so designed that, should one span be removed, it would not be tipped over by the totally unbalanced pull of the cables of the remaining span.

The superstructure of the West Bay crossing is not greatly different from that of other long suspension bridges. Some of the pertinent data are given in Table I.

TABLE I. PRINCIPAL DIMENSIONS AND QUANTITIES
San Francisco–Oakland Bay Bridge

LENGTH OF MAIN SECTIONS	
San Francisco terminal of west anchorage	4,200 ft
West Bay crossing	10,450 ft
Yerba Buena Island section	2,950 ft
East Bay crossing to Toll Plaza	19,400 ft
Toll Plaza to Oakland terminal	6,500 ft
Total	43,500 ft, or $8^1/_4$ miles
DATA ON WEST BAY SUSPENSION SPANS	
Length of main spans	2,310 ft
Length of side spans	1,160 ft
Dead load of main span (two trusses)	19,100 lb per lin ft
Dead load of side span (two trusses)	20,100 lb per lin ft
Live load	7,000 lb per lin ft
Dead load for utilities, additional	500 lb per lin ft
Main cables (two)	$28^3/_4$ in. diameter
Sag of main cables	$^1/_{10}$ span
Diameter of wires	0.195 in.
Wires per strand	472
Strands per cable	37
Towers, above water	477 and 522 ft
Piers, below water	100 to 210 ft
Height of center anchorage, above water	294 ft
Clearance, minimum	200 ft
DATA ON EAST BAY CROSSING	
Arms of cantilever span (each 412 ft)	824 ft
Suspended span	576 ft
Total	1,400 ft
Anchor arms, each	508 ft
Clearance of cantilever span	185 ft
Through trusses; five spans, each	504 ft
Deck trusses; 14 spans, each	288 ft
STRUCTURAL QUANTITIES	
Structural steel	152,000 tons
Cable wire	19,500 tons
Reinforcing steel	17,000 tons
Concrete	1,000,000 cu yd
Timber	30,000,000 fbm

A feature of the design which warrants some comment is the location of the San Francisco cable anchorage some 900 ft beyond the end of the side span. Solid rock was found at such a depth at Pier W-1 that it was more economical to move the gravity-type anchorage to the west, where rock could be reached at a more favorable elevation. The addition of 900 ft of unloaded cable will have the adverse effect of making the west suspension bridge more flexible and consequently subject to greater deformations under the action of live loads. Tower W-2 is particularly affected and is reinforced for a large part of its height to provide for the excess bending stresses. It is designed for a total extreme movement of its top of 6 ft 10 in. in the direction of the bridge.

A rather unusual specification has been adopted for the steel work on this part, as well as all other parts of the work. The contractor is required to store all fabricated material on skids in a place subject to the action of the elements for from three months to a year. It is anticipated that this will cause the steel to rust and thus

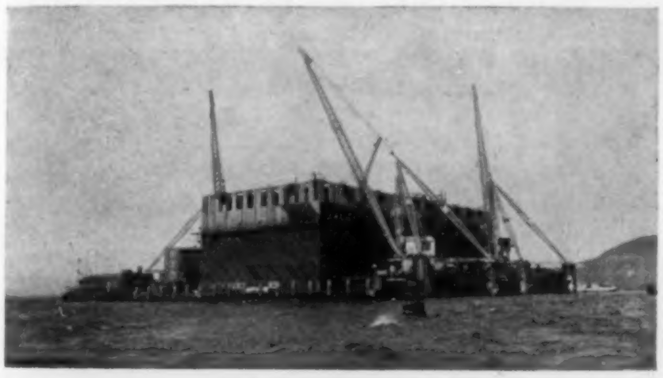
loosen a very large part of the mill scale. At the end of the storage period the steel surface will be sand blasted to a gun-metal finish and the paint applied. It is estimated that 200,000 gal of paint will be needed.

In the east channel the water does not exceed 40 ft in depth, and over most of the distance the depth is much less. Since rock lies at depths generally over 300 ft, all the piers beyond the island either are founded in strata of consolidated sand and gravel containing some clay, or are carried on timber piles.

Piers E-3, E-4, and E-5 being of similar construction, a description of E-3 will be typical. In plan, this pier measures 80 ft 1 in. by 134 ft 6 in. and is carried to the record depth of 225 ft. The contractor chose to employ the open-well caisson type of construction using false bottoms in the wells to provide flotation until the caisson landed on mud. While floating, the caisson was maintained in position by heavy steel spuds driven deep into the mud bottom. It has 28 rectangular wells, each 15 ft $7^1/_2$ in. by 15 ft 5 in., of heavily reinforced concrete. It was felt that the weight of the caisson might prove insufficient to overcome skin friction when the greater depths were reached, and for this reason an elaborate jetting system was incorporated in the walls with outlets along the cutting edge.

Beyond Pier E-5 there will be 17 piers, each supported by from 300 to 600 untreated timber piles. In constructing these piers the mud will be excavated from within a steel sheet-pile cofferdam to about elevation −50. The piles will then be driven, and a blanket of gravel 5 ft deep will be placed over the entire pier site. This blanket will act as a bed for the concrete seal. After the seal has hardened, the cofferdam will be unwatered and remaining construction done in the dry.

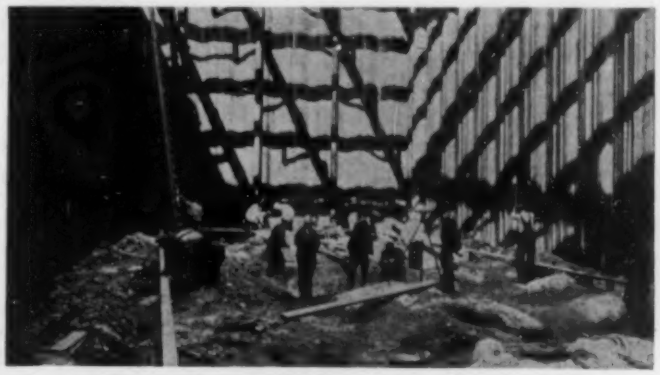
The outstanding feature of the East Bay superstructure design is the 1,400-ft cantilever span adjacent to Yerba Buena Island. Somewhat of a departure from usual design was made in the method of supporting the suspended span. Normally the suspended span is held in place by eye-bars joining the top end panel point of the cantilever arm and the lower end panel point of the suspended span. In this case, however, the lower end panel point of the cantilever arm will be directly pin-connected to the lower end panel point of the suspended



Caisson for Central Anchor Pier in West Channel
Sides and Domes of Flotation Cylinders for World's Largest Caisson Being Raised

span. The purpose of this is to stiffen the structure and permit transmission of longitudinal loads to Pier E-1.

Since the structure is in a region which has suffered, and may again suffer, more or less violent earth shocks, unusual precautions have been taken to safeguard against this hazard. All elements of the bridge are designed for an acceleration of the supporting material of 10 per cent that of gravity. It was readily recognized that the usual criteria for earthquake design would not be satisfactory in dealing with this structure. In view of this, an exhaustive study was made and design methods were



Inside the Sheet-Piling Cofferdam for Pier E-20

evolved which took into consideration the various peculiarities of the problem.

In the case of the channel piers, the horizontal force created by the acceleration of the mass will be augmented by forces due to the movement of the pier through the water and the soft mud immediately below. In fact it is conceivable that the soft mud may have an acceleration of its own in a direction opposite to that of the pier. These forces were incorporated in the analysis. In dealing with the superstructure, and particularly the suspension spans, the elastic and mechanical flexibility of the elements were fully considered. For earthquake design a 40 per cent increase in basic unit stress was permitted.

It is estimated that the period of construction will be four years. At the start, a toll equal to the ferry fare will be charged. This will be reduced gradually over a period of 20 years, after which the bridge will become free. Traffic estimates for 1937, the year the bridge will be opened, are for 8,000,000 vehicles carrying an average of two passengers each and an additional 35,600,000 passengers carried by the interurban electric system. By 1950, according to present estimates, these items will be increased to 25,000,000 automobile passengers carried in 12,600,000 vehicles, and 40,000,000 interurban passengers. The total estimated cost is $77,250,000, which includes $6,600,000 for approaches and $15,600,000 for the facilities to be provided for the interurban electric railway system.

PERSONNEL

The bridge is being built for the California Toll Bridge Authority, of which Governor James Rolph, Jr., is chairman, by private contractors under the State Department of Public Works, of which Earl Lee Kelly is director. In the San Francisco–Oakland Bay Bridge Division of the Department of Public Works, C. H. Purcell, Assoc. M. Am. Soc. C.E., is Chief Engineer; Charles E. Andrew, Bridge Engineer; and Glenn B. Woodruff, Engineer of Design, both Members Am. Soc. C.E. The Board of Consulting Engineers consists of Ralph Modjeski, Chairman; and Daniel E. Moran, Carlton S. Proctor, Leon S. Moisseiff, Charles Derleth, Jr., and H. J. Brunnier, all Members Am. Soc. C.E. Professor A. C. Lawson is the consulting geologist. The Consulting Board of Architects consists of Timothy L. Pflueger, Arthur Brown, Jr., and John J. Donovan.

Directing a CCC Camp in Michigan

Problems and Accomplishments Over an Eight-Month Period Recorded

By J. W. Orton

Associate Member American Society of Civil Engineers
Camp Superintendent, Camp S. P. No. 1, Hayes State Park, Onsted, Mich.

PROBLEMS hitherto unknown are facing the engineer today. Many, like those here presented, relate to community rather than personal economic questions. With the increased amount of leisure to be enjoyed if the "New Deal" achieves its anticipated success, people will more and more clamor for parks in which profitably to spend their leisure. Hayes Park, whose development by the Civilian Conservation Corps (CCC) is described in this article, will objectively answer such a real need. The problems presented were various and not in themselves complicated. But because they are representative of a vast effort throughout the country—an effort in which the engineer has played an outstanding part, as in this instance—they deserve to be chronicled. The period covered is eight months, ending March 1, 1934, and the aggregate of work reached the impressive total of over 140,000 man-hours.

HAYES State Park, in the Irish Hills of southern Michigan, 60 miles west of Detroit on route U. S. 112, is one of the newer and undeveloped areas in the Michigan park system. It is, however, well located, close to the center of population of the lower peninsula, easily accessible by road, and in the center of a picturesque rolling country, glacial in origin and studded with numerous small lakes. Consequently the State Park Department selected it as one of three of its parks in the lower peninsula in which to locate a Civilian Conservation Corps Camp during the first enrolment period and has continued its operation in the second and third periods.

During the season of 1933 it was visited by well over six hundred thousand people. When it is fully developed and becomes well known it should prove one of the finest state parks in the country and should draw large throngs of people.

To develop a state park is a problem involving the altering of terrain so as to preserve natural beauty and at the same time make the park serviceable and enjoyable

Deep Cut for Road on East Side of Hayes Park

to its visitors. Many problems of landscape architecture and engineering are involved, each park having its own grist of projects to be analyzed and approved before work can begin. These may be initiated by the Camp Supervisors, the State Park Authorities, or the National Park Representative. Before a construction project can proceed, plans must be drawn and approval given by the Superintendent of State Parks and the District Officer of the Department of the Interior, and plans must be submitted to Washington. A master map showing the entire park with all work projected is of prime importance and is intended as a progress guide for the developments. Fortunately Hayes Park previously had been surveyed and mapped with a 2-ft contour interval. The tract contains 475 acres, including all of Round Lake (70 acres) and approximately a half mile of shore on Wampler's Lake (Fig. 1).

Work is well along on many major projects in Hayes Park. Aspects of each type and methods of carrying on operations with Civilian Conservation Corps labor will be given.

Park Roads Present Some Problems

In state parks, a high-grade road is considered more or less of a detriment, marring the natural beauty and tending to attract large crowds merely as sightseers. Hayes Park abuts on a main highway, U.S. 112, but its heavy traffic does not interfere with park travel. The park entrance road is a state highway, M 124, of concrete 20 ft wide. It traverses the full length of the park, about a mile and a quarter, with gentle winding curves on easy grades. The new construction therefore involved simply about $1^1/_4$ miles of graded gravel roads 22 ft from shoulder to shoulder, with 3-ft ditches. Fills and cuts were balanced and much thought was given to harmonizing the road with the adjacent terrain.

These roads were located with the intention of providing easy access to the grounds but not of intruding too

A Typical Cobblestone Gutter

conspicuously. Narrow, unimproved trails and footpaths cover the wooded and scenic areas. Gravel roads loop into the west and east sides of the park. In some cases these have involved merely the widening and improving of existing trails; in others, the making of cuts and fills as much as 8 ft deep.

Side drainage and cross-road drainage have been given prime consideration. Where erosion of ditches was thought to be inevitable, they were dry cobbled, thus adding to their pleasing appearance as well as to their permanence.

The soil in the park is of glacial origin, a heterogeneous yellow clay, sand, or gravel with no regularity as to deposition. Cobbletones abound both in the park and the nearby farms, so that there is no dearth of such material for gutters, walls, and beach improvement. The gravel, actually a coarse sand containing a few larger pebbles, is not very good road material; however, the roads last quite well with but little maintenance and occasional additions of small pebbles. A small amount of black soil or loam is found in some places in the park, and such areas are used as sources of supply for seeded ground, or for top soil on gravelly areas which are to be planted. Crossings of the peat swamps have given considerable trouble, because the gravel settled through to the solid ground. One road has been filled to grade four times in as many months. However, the peat has its uses as a soil lightener in yellow clay.

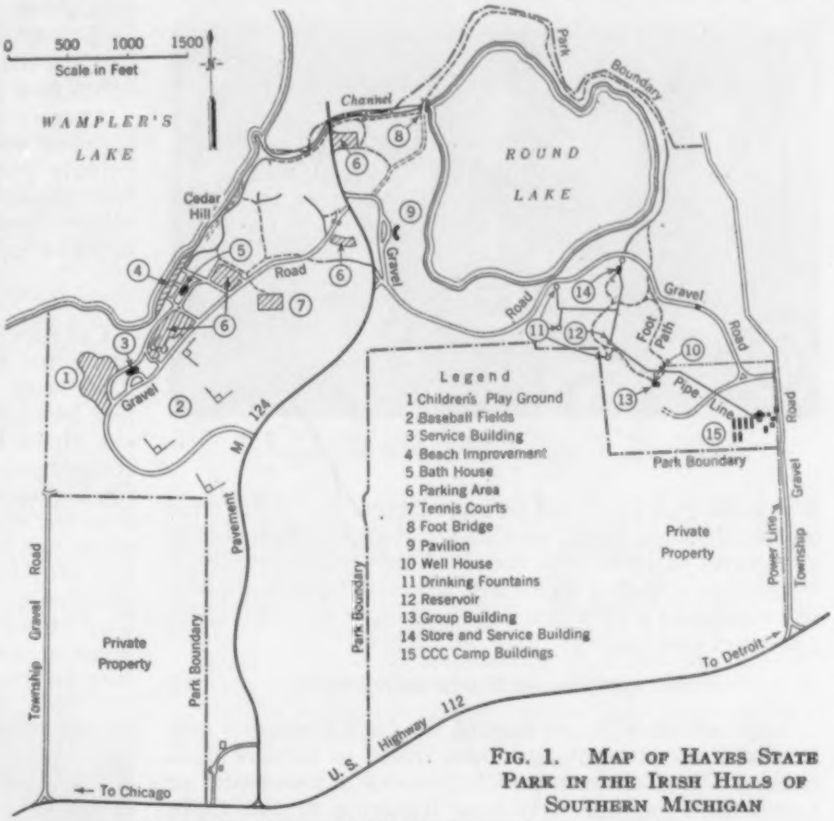


FIG. 1. MAP OF HAYES STATE PARK IN THE IRISH HILLS OF SOUTHERN MICHIGAN

Extent and Nature of CCC Improvements Undertaken

BANKS PROTECTED FROM EROSION

Where the park borders on Wampler's Lake, for a distance of approximately 800 ft, there is a steep bank of typical clay and gravelly soil extending from the water's edge upward at an angle of about 45 deg to a maximum height of 80 ft. This bank, which had been badly eroded and covered with dead and dying timbers, was completely protected by "stepping" its slopes with timbers securely staked. The horizontal spaces between timbers were made of sufficient width and depth to support plant life and grass. Small shrubs were placed on the slopes.

It is the intention that the plant life will take root and form a bank of uniform slope tending to resist erosion long after the timber riprapping has rotted. This barren hill has long been an ugly scar on the landscape when viewed from across the lake. The improvement should not only benefit the park by protecting the hill as a natural beauty spot but should also improve the view from the cottages which dot the entire shore.

At the top of the steep bank overlooking Wampler's Lake is a promontory known as Cedar Hill (Fig. 1). From this vantage point the view, especially at sunset, is worth traveling miles to see. To preserve it and to encompass two oaks which are well-known landmarks, a wall of vertical logs has been built, of sound oak recently cut in the vicinity and varying in length from 12 to 25 ft, the longer ones being toward the middle of the curve. The 12-in. butts were buried 4 ft in the hillside; the tops were sawed off at a uniform elevation slightly above the normal ground level at the top of the hill. After the logs were peeled and seasoned, each was given two coats of hot creosote. They were arranged in nearly a perfect arc and were fastened in place by seven 1-in. galvanized standing ropes, each about 125 ft long and containing a galvanized turnbuckle. The ropes encircle the logs and extend back into the bank on tangents for a distance of 40 ft, to where they are secured into a poured reinforced concrete "deadman," 2 by 2 by 8 ft. When the logs were in their correct position each cable was brought to proper tension by tightening the turnbuckles, so that each of the seven cables carries its share of the load. A $^1/_4$-in steel plate prevents the cables from biting into the wood;

TRANSPLANTING SMALL CONIFERS

MOVING A LARGE TREE WITH A TREE WAGON—A WINTER JOB

to prolong their life they were treated with a primer and one coat of metal preservative.

It is expected that the site will be ornamented so that it will be one of the beauty spots of Michigan. Climbing vines will be planted at the base and trailing vines at the top, so that eventually the wall will be obscured.

COMPLETED RESERVOIR OF 10,000-GAL CAPACITY

A wooden guard rail will be built around the entire crest of the hill, some rustic benches set at appropriate points, and gravel paths built to the site from several directions. Ultimately a shelter house will be erected nearby, which will command a view of a large section of the Irish Hills, a scene of rare beauty.

CULTIVATING OLD AND NEW TREES

Approximately 50 per cent of the park is wooded area in which no attempt had been made to remove dead timber. Until the leaves fell, a crew of 18 men under an experienced foreman were kept trimming surplus limbs and dead wood and removing numerous dead trees. A great improvement in the appearance of the trees is noted.

Several years ago about a thousand small deciduous trees were planted in the park. Many of these were dying due to lack of moisture, especially during the past two years. A cultivation crew was organized and spent the entire summer in loosening the ground around the base of all these trees and frequently watering them from a tank wagon. By reason of this protection, very few of them died.

A park planting map has been prepared showing where new trees and shrubs are to be placed. For special areas around buildings and for beach and parking areas, additional detailed maps will ensure artistic and seasonal appearance. Roadside banks have been planted to protect them from erosion and from the destructiveness of picnickers as well as to enhance their appearance.

As soon as the fall planting season arrived, a planting crew was organized. Local coniferous trees were purchased and set at points of vantage. Ten thousand white cedars were planted in swampy areas. Small trees and shrubs were obtained free of charge, whereas larger trees were purchased nearby. The latter were dug and balled early in the season and moved into the park on a tree wagon. These trees were all well wrapped and guyed under the direction of a landscape architect. In a few years they will be greatly appreciated by park visitors. In addition, several thousand small trees and shrubs are scheduled for purchase and planting during the spring of 1934.

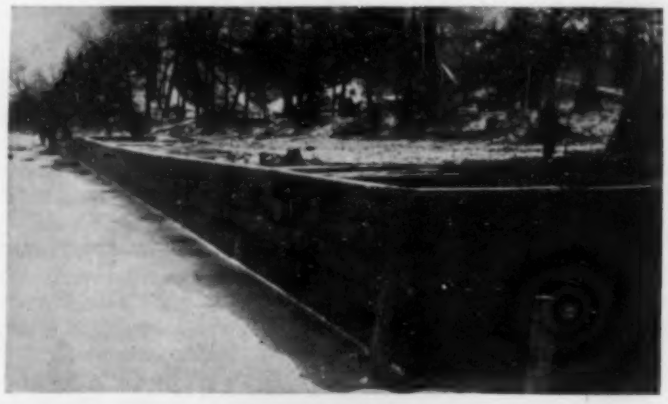
In all, 1,600 lb of grass seed was purchased, most of which was sown during the fall. A few hundred pounds is left for spring seeding as the denuded areas become apparent.

Sodding of roadside banks has been one of the most conspicuous improvements in the park. Four large banks which formerly were steep, ugly gashes through hills, serrated with gullies, have been trimmed and rounded to gentle slopes which blend well with adjacent areas. These have been covered with sod cut from the baseball diamond. With small trees and shrubs, these banks now present a very pleasing appearance.

At one point, the view of a scenic swamp area from the highway was obstructed by a hill. This hill has been entirely leveled to a gentle slope, which will be seeded, thus presenting a pleasing lawn as a foreground to the colorful swamp. The excavated material proved useful as fill for roads and parking lots.

PARKING AREAS

Car parking areas have in the past been entirely inadequate for the enormous Sunday and holiday crowds. They were in fact merely irregular staked fields. The four new parking areas proposed will accommodate over one thousand cars. These are in sections of greatest congestion, two on each side of the bathhouse, one near the highway in the picnic area, and one on the highway near the camping area. At least one, and perhaps another, will be completed before summer. These lots are laid out with alternate 50-ft parking spaces and 10-ft planting areas, the latter surrounded by concrete curbs, 6 by 24 in., which are set in a 6-in. layer of gravel so that water in the road will seep through, thus providing moisture for the vegetation. Much grading must be done;



BREAKWATER BUILT OF OLD RAILROAD TIES, A BEACH IMPROVEMENT ON WAMPLER'S LAKE

in some places a cut of 5 ft is required to bring the lot to proper elevation before it is covered with 4 in. of small gravel and rolled. The areas will be carefully landscaped, and it is also proposed that a system of floodlight illumination be installed.

ELEVATED BATHING BEACH BEING BUILT

The bathhouse, on the east shore of Wampler's Lake, has never enjoyed the popularity that it should, because the beach is more or less muddy and stony near the shore line. No dry, sandy area for children's play or for resting in the sand was available although from a point 40 ft from shore a fairly level shallow and sandy stretch extended several hundred feet into the lake. Consequently an artificial sandy beach elevated above the present water level and extending out into the lake about 40 ft is being constructed, retained by a timber breakwater with a tight front wall 5 ft high and 600 ft long.

192 CIVIL ENGINEERING for April 1934 VOL. 4, NO. 4

This is being built of old railroad ties in the form of a crib 8 ft wide with cross walls 8 ft on centers. The ties, all oak and steam-creosoted, were purchased from a recently abandoned stretch of railroad. They are being fastened together with $^3/_8$ by 10-in. wrought-iron spikes and are weighted down with rock, graduated in size from large boulders at the bottom to small pebbles at the top. It is intended that the weighted crib will be strong enough to resist ice action, which is rather destructive on this particular shore. Stairways with handrails will be spaced every hundred feet for access to the water.

Filling for the interior space will be of ordinary earth to a height that will leave an 18-in. depth of sand. This sand, obtained locally at reasonable cost, will be finished on a uniform grade extending from the top of the breakwater to an existing lawn along the beach. A sharp line of sod will be maintained between the beach and the planted area. Plans are afoot for a system of flood lighting, also for planting and landscaping along the shore, so that the beach should prove to be the most popular part of the park in years to come.

Already the park has a network of footpaths, some merely of dirt, others, near the public buildings, of gravel and asphalt. These are to be extended. The trail surrounding Round Lake is to be improved for a foot or bridle path. For the point where the trail crosses the lagoon, a timber bridge is planned.

Cedar Hill, overlooking Wampler's Lake, had several stairways up its steep slopes. These were badly rotted and generally dilapidated and have been supplanted by new ones of creosoted railroad ties.

Completed Group Camp Building

On its western side the park was partially supplied with water; in fact there were two separate systems, one of well water for drinking and the other of lake water for toilets and sprinkling. The eastern side, however, was without a water supply except for two hand pumps. As this area will be used exclusively for campers, an adequate system was needed quickly. Consequently a well 36 ft deep, cased with 4-in. pipe, was drilled approximately in the center of the area. When the water was analyzed, both quantity and quality were found to be good. A 70-gpm, motor-driven, horizontal, centrifugal pump was installed in a pump house 8 ft in all dimensions, with concrete floor and pump base, concrete-block walls, and a wooden roof, tin covered. The discharge is directly into a 10,000-gal concrete reservoir at the summit of a hill 500 ft away and 70 ft higher, furnishing pressure to all points on the east side of the park. The operation of the pump is automatic and provides for a 4-ft variation in high and low levels.

From the pump the discharge line is $3^1/_2$ in. in diameter; the main feed line out of the reservoir is 3 in.; and sizes to points of use vary between $^3/_4$ in. and 2 in. The pipe is all galvanized, and joints have been coated with a bitumastic compound to increase longevity of the line. Bubbler fountains, to be scattered throughout the camping area, will be substantial and attractive, with a

Typical Mattress of Boughs Sunk Through the Ice as a Fish Shelter
Forty-Five Such Shelters Sunk in Two Lakes

bucket-filling attachment, and will be well drained and easy to maintain. The water is also piped to two group camp buildings and to a store and comfort station. During the winter the entire water supply of the CCC camp was from this system, which was connected with the camp by a temporary 2-in. line.

In all, 2,525 ft of distribution pipe has been laid to date, mostly at shallow depth, since the system is not intended for winter operation; but frequent deep cuts of 5 or 6 ft have been made in hills.

PERMANENT STRUCTURES BEING INSTALLED

A number of structures are already planned or built. The group camp building, in a secluded area on the eastern side of the park, is primarily for the use of groups up to 50 boys or girls who make extended visits. A store and comfort station, centrally situated in the camping area, is of concrete construction in the basement and first floor, and of wood in the superstructure. The exterior of the building is faced with cobblestones with pointed joints. Sewage will discharge into a septic tank, thence into a dosing chamber, and finally into a buried clay-tile distributing system. The Pavilion, located centrally in the park along the concrete highway, will be a permanent concrete and cobblestone structure blending with the wooded surroundings. The Shelter House, proposed for the top of Cedar Hill, is not yet designed.

SPORT TO BE ENCOURAGED

In order to improve the two lakes for fishing and protect the young fish from the larger predatory ones, shelters, 6 by 10 ft, made of mats of poles and tree branches were sunk during the winter in water 8 ft deep. These were securely wired and weighted with sand bags, dragged out on the lake on a sled, and sunk through a hole cut in the ice. Twenty such shelters were placed in Round Lake and twenty-five in Wampler's Lake.

A hard baseball field, three playground ball fields, tennis courts, horseshoe and shuffle-board areas, and children's playground apparatus are provided. The site of the hard baseball field was originally selected because it was the largest nearly level area in the park. The sod has been stripped and used on roadside beautification, and a start has been made on grading. This will be done to standard league specifications, and the infield

will be seeded with specially selected grass. It is contemplated that ultimately a small bleacher and comfort station will be built.

DISTRIBUTION OF LABOR

Beside the improvements mentioned, considerable labor was expended on other minor operations. Neighborhood fires were fought as they occurred; one, a peat-bog fire, raged for four days. The park lagoon was partially dredged of muck in its shallow stretches; much nursery work and ditch-cleaning work throughout the park were undertaken; and a garage and supply house for work operations were built. Of course the maintenance of the truck fleet, tools, supplies, and equipment, and the clerical work required much labor time not included in project work.

LAYING SMALL PIPE LINE IN CAMPING AREA

From the start of the camp on July 5, 1933, up to March 1, 1934, the man-hours of work distributed among the projects were as follows: road construction, 51,000; parking areas, 17,000; cultivation and trimming of trees, 14,000; seeding and sodding, 11,000; bathing beach, 11,000; and water system, 10,000. Shorter periods were devoted to the other activities. Included in these totals are the noonday lunch periods, which occupied at least an hour, thereby cutting down the actual work hours shown by $12^1/_2$ per cent. The grand total reached 142,000 man-hours for the whole period.

PROPOSED BRIDGE OVER A LAGOON NEAR ROUND LAKE

To a contractor these totals would represent a staggering amount of labor. It must be borne in mind, however, that many of the workers were mere boys under 20 years of age, who had never been engaged on useful work of any kind and who scarcely knew which end of the shovel to use when they started. Very few with actual construction experience were found; many have developed into good workmen, however, and further improvement is expected if the camps continue.

Another aspect of the matter is that the entire CCC work is to be considered, partially at least, as a humanitarian and welfare project. The men are not pampered, neither are they driven as a contractor would drive older, more mature men. The pay is relatively small, and effort is not made to place the output of work on a production basis such as large contractors would employ. The use of machinery on the work is frowned upon—hand labor is the order of the day. When it is realized that all the earth was moved with four small trucks, wheelbarrows, picks, and shovels, the output is not discouraging. Then too, work crews were ordered out on bad days in order to complete their required hours per week. In many cases these operations were conducted when the ground was muddy or frozen. If the same work had been done by a contractor, to show profit, it would have been postponed until better weather.

EROSION WORK ON STEEP HILLSIDE

With the writer in charge, the field work has been carried out under the supervision of nine foremen, the majority of whom are technical graduates. The State Park Department, of which P. J. Hoffmaster is Superintendent, together with the U. S. Department of the Interior, of which G. W. Olcott is Inspector, have given approval to projects and have been in constant touch with all phases of the work. The Second District of the U. S. Department of the Interior Office of National Parks, Buildings, and Reservations, in charge of P. V. Brown, at Indianapolis; and the Washington office of the Emergency Conservation Work under Robert Fechner, Director, have approved plans, received reports, and had executive authority over all operations.

As regards cost of operations—including supervision, materials of construction, supplies, tools and equipment, and skilled labor payrolls—this is borne by the Department of the Interior with CCC funds, which department retains ownership of non-expendable property. The writer has never had an opportunity to investigate operations in any other camp, but if in the more than 1,500 CCC camps throughout the country a similar amount of work is being performed, the sum total of such useful and permanent effort is tremendous. The CCC program, in its threefold aspects of productive work, welfare, and upbuilding of the Nation's youth, is unquestionably one of the most worthy of the programs advanced for improving economic conditions.

194 CIVIL ENGINEERING *for April 1934* VOL. 4, NO. 4

River Hydraulics in Czechoslovakia

National Laboratories Conduct Model Experiments for Irrigation, Flood Control, and Power Structures

By Donald P. Barnes

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During March 1933, while studying in Europe as a Freeman Traveling Scholar of the Society, Mr. Barnes was privileged to see the progress that the Republic of Czechoslovakia is making in the nation-wide development of its rivers. Storage is needed to augment the low flow for navigation, and to prevent the annual inundation of farm lands by floods. Many power sites have already been developed and designs for others are being investigated by means of models in the national hydraulic laboratory. The Republic's first concrete dam, at Vranov on the Dyje River, now approaching completion, incorporates several features which were first developed by model study in the laboratory.

In the 16 years that have elapsed since its inception, the Czechoslovak Republic has not been slow to attack its complex problems of river regulation. Formerly, when it was a part of the Austro-Hungarian Empire, its rivers were administered from Vienna, but since 1918 it has had to organize a hydrographic bureau and to build its own laboratory for testing river structures.

Czechoslovakia occupies a peculiarly central position at the headwaters of three of Europe's important navigable streams, the Oder, the Elbe, and the Danube (Fig. 1). In the more distant past even more than in recent years, river traffic has been an important factor in the economic life of the area; and the control of the not infrequent floods that discharge into the flat valleys both within and below its boundaries remains a serious problem.

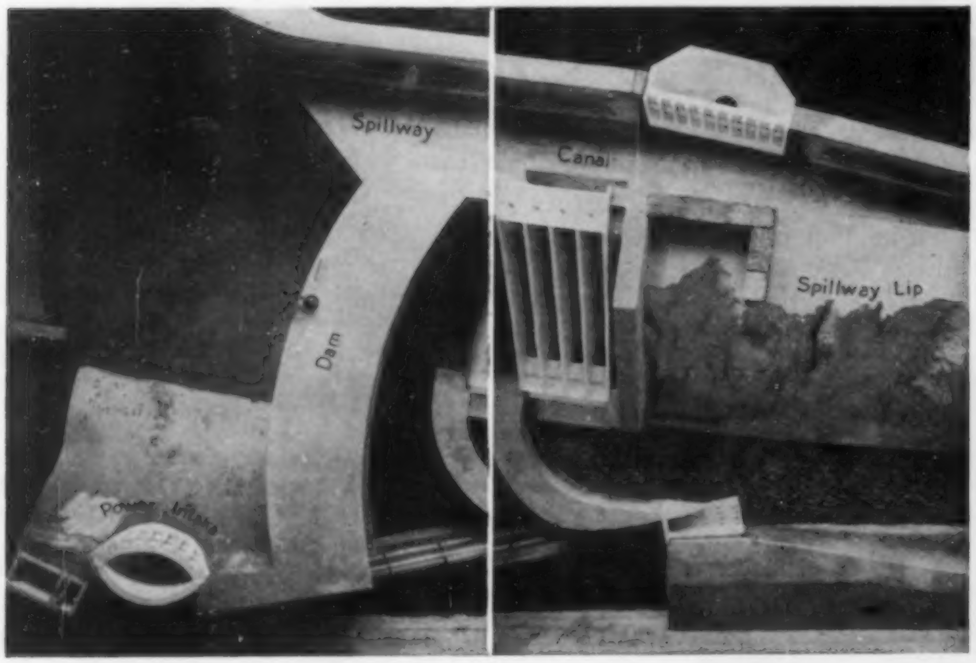
A commission established by the Diet of the Kingdom of Bohemia in 1875 concerned itself with studies of rainfall and run-off from the upper basin of the Elbe. It is of interest to note that the state of "Cesky," or Bohemia, corresponded almost exactly to the drainage basin of the Elbe, shown in Fig. 1. Within the course of a few years, it succeeded in establishing a network of rain- and stream-gaging stations rivaling in number those of any other similar area in Europe. In 1889 the work of the commission was delegated to the department of agriculture, which in 1892 instituted the daily water stage forecast service which has proved of such value to the relatively slow-moving river freight.

After the formation of the Republic, the National Hydrological Institute was established, with divisions at Brno (Brünn), Opava, Bratislava, and Užhorod. It was assigned the triple function of accumulating and publishing statistics of precipitation and run-off, of analyzing and preparing plans for such works as flood control dams and training levees, and of conducting various experimental researches on problems of river regulation.

To serve the organization in accomplishing the first of these tasks are some 1,720 rain-gaging and 650 stream-gaging stations. The statistics gathered are made public in graphical and analytical form in a monthly hydrological bulletin. A "Monthly Résumé of Meteorological Observations," from the National Meteorological Institute, is also included. In addition the institute publishes a year book giving 25 and 50-year averages, totals, and other pertinent data.

Perhaps the most important activity of this branch of the institute's work is its water-stage forecasting service. Daily reports are received by private telephone from nearly forty stations along the Elbe and its tributaries. From these, the flow at Děčin and at Ústi, important

Fig. 1. Czechoslovak Rivers Connect with Three Seas by Way of the Elbe, the Oder, and the Danube



MODEL OF PROPOSED ŠTĚCHOVICE DAM, VLTAVA RIVER, CZECHOSLOVAKIA
Tested at T. G. Masaryk National Hydrotechnic Institute

shipping terminals near the German border, can be accurately predicted 24 hr in advance. During critical storms, reports are received hourly or oftener from each of the stations. A similar arrangement has been made in cooperation with the Austrian and Hungarian authorities to report the behavior of the Danube's tributaries.

MODEL SPILLWAY OF ŠTĚCHOVICE DAM DISCHARGING EQUIVALENT OF 56,500 CU FT PER SEC
Tailrace of Power Plant Was Located Downstream in Quiet Water

In performing its second function, the Hydrological Institute has made studies of the entire field of possibilities for the development of power and irrigation and of improvement works for navigation and flood control.

DEVELOPMENTS ON THE ELBE

Dečin is the present lower limit of navigation on the Elbe. The proposal to open the channel through the narrows to Germany is opposed both because of the expense of the improvement itself and the summer water shortage, the flow varying between 1,400 and 197,000 cu ft per sec at Dečin. On the Elbe and Vltava rivers the heaviest barges can travel as far upstream as Jaroměř and Praha (Prague), respectively. Plans for the canalization of the Vltava are already complete, and much of the work from Praha to Cesky Budějovice has been finished despite the difficulty that the low-water flow at Stěchovice is only 250 cu ft per sec. Plans have also been considered for the extension of a navigable channel from the Elbe up the Ohře River as far as Cheb. This route would penetrate a great manufacturing and coal mining area now served only by rail. Another projected plan is the improvement of the Berounka River from the Vltava to Plzeň (Pilsen), also an industrial center and known as the home of the world-renowned beer.

Closely connected with these plans are the projected dams to regulate flow and furnish power and irrigation water. Some of these are shown on the map, Fig. 1. The dams at Slapy and Stěchovice, which will be built in the near future for controlling the Vltava River, will be mentioned in more detail in connection with the experimental work at the T. G. Masaryk National Hydrotechnic Institute. Near Karlsbad, a dam is now under construction which will assist in the regulation of the Ohře, and other important dams are already completed or under consideration in the basin of the Elbe. In order to ensure sufficient flow through the summer for navigation below Dečin, every possible storage site would have to be utilized.

IMPROVEMENTS ON THE DANUBE RIVER

The Danube has long been open to river traffic far above the Czechoslovak boundary, but its four largest tributaries from the eastern slopes are as yet only usable by lighter craft. The Morava River is the most significant of these because of its potential connection by canal with the Oder to the north. Such a link between the Black and the Baltic Sea has been considered for many years, and now awaits only the return of prosperity to receive its final impetus. It would furnish a double outlet for Moravian coal and iron, and would give the

UNEXPLAINED STANDING WAVES AT CREST OF ŠTĚCHOVICE SPILLWAY MODEL

nation a strategic location on another international trade route. The already completed Bystřička Dam would be available to supply part of the needed storage.

Plans for the improvement of long reaches of the Váh, the Hron, and the Tisa rivers are also being considered. All these imply the construction of dams to impound and regulate run-off.

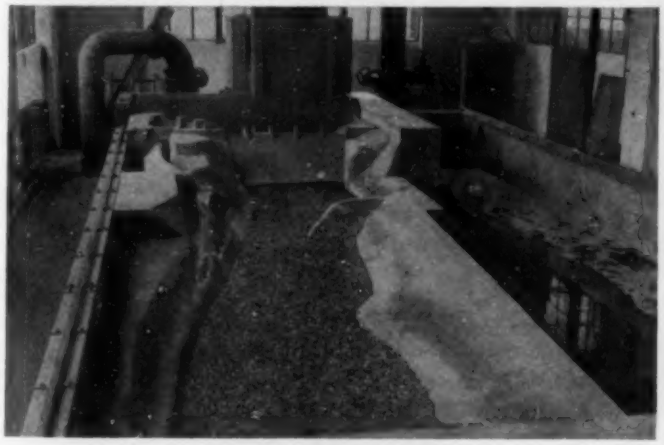
NATIONAL HYDRAULIC LABORATORY AT PRAHA

The experimental phases of the department's work were greatly facilitated when the T. G. Masaryk National Hydrotechnic Institute was opened at Podbaba near Praha in 1930, on the occasion of the birthday of the Republic's president. The hydraulic, as distinguished from the hydrographic division occupies one of twin buildings, of which the second is not yet completed. The forepart of the building comprises four stories of office and shop space, and the rear section contains two laboratory floors. In addition to these, a 500-ft channel is enclosed in the structure.

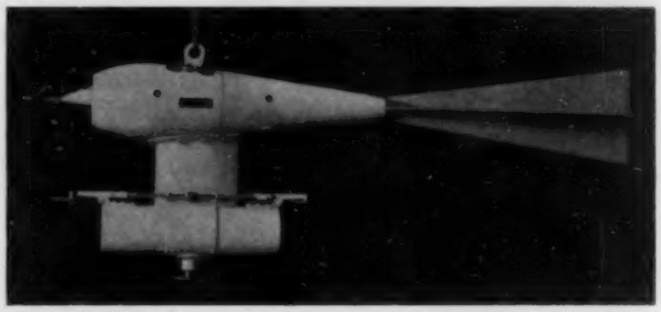
It is evident that the laboratory has been carefully planned. It is equipped with two glass-walled flumes, one 14 in. wide, the other 27 in. wide and appropriately deep, in which studies of weirs, tumble basins, and spillways can be readily conducted. Models of entire dams or of stretches of canals and rivers can be erected in a concrete channel 16 ft 3 in. wide. Pumps, measuring weirs, photographic apparatus, including motion picture instruments, the constant-head reservoir, and all the necessary auxiliaries of the modern hydraulic structures laboratory are sufficient for every purpose.

Typical of the kind of work which the institute is called upon to perform are the investigations recently undertaken in connection with the development of the Vltava River. In order to determine the relative merits of three alternative proposals, the hydraulic adequacy of spillways, intakes, and energy-absorbing devices had to be studied experimentally. For this purpose models were erected in the river channel, their capacities and behavior were observed, and in some instances valuable changes were suggested.

A model of one of the dams proposed for the Stěchovice site is illustrated. This model is for an arched dam having a spillway canal cut into the rock at the left of the canyon, discharging laterally over the mountain side into the river bed below the dam. At the right of the canyon a roughly elliptical forebay with seven openings leads the water into the power plant penstock. The two additional structures shown at the sides of the spillway canal represent abandoned suggestions for the power intakes. The maximum expected flood is 84,600 cu ft per sec. In the model the capacity of the spillway alone was found to be equivalent to 70,600 cu ft per sec and that of the penstock to 35,300 cu ft per sec, as designed. The combined discharge from the tailrace and spillway of the model was equivalent to 85,000 cu ft per sec, causing a turbulent condition below the cascade, which necessitated the relocation of the power plant downstream in quiet water. The velocity at the foot of the fall corresponds to 88 ft per sec.



Model of Another Design for Štěchovice Dam
Spur Dike at Right Designed to Reduce Bank Erosion



Combined Current Meter and Sediment Sampler
Developed by Czechoslovakia's National Hydrotechnic Institute

A model of an alternative design for the Stěchovice Dam is also shown in a photograph, which conveys an excellent conception of its arrangement in the lower end of the channel of the laboratory. The experiments indicated that the forebay would be entirely inadequate. Its choked state was clearly indicated by the model. At the same time the outlet pipe was only half filled. Although the penstock was designed to carry 28,300 cu ft per sec, only 14,000 cu ft per sec would pass through it under the head that would be available. A satisfactory spur dike was developed to protect the left bank from erosion under the impact of the flood from the central overflow spillway.

Model Used to Develop Best Form of Stilling Basin for a Dam on the Blanice River

In connection with the work of its gaging stations, the institute has developed a combination current meter and sediment sampler. A fuse of a soluble salt holds the two disc flap valves open at the ends of the sampling cylinder for a period which may be fixed by changing the solubility of the fuse. When the fuse is finally dissolved, generally after three minutes, springs are released and the discs snap shut, imprisoning the desired sample. As an added convenience, a plunger contact is provided below the cylinder to indicate the arrival of the meter at the river bottom.

Recently the scope of the institute's experimental activities has been enlarged to include limited investigations in the field of soil mechanics as it concerns river structures. Small high-pressure cylinders after the design originated by Terzaghi have been used to examine the permeability of dike materials. At present a study of the efficacy of various artificial mixtures for injection into soils to increase their impervious qualities is being undertaken with the object of developing a product suitable for use in levees and canal embankments.

HYDRAULIC STRUCTURES LABORATORY AT BRNO

At the Czechoslovak Technical University at Brno, the older hydraulic structures laboratory established in 1917 is also actively engaged in experimental work for the state. Perhaps its most spectacular tests and those most immediately applicable were the ones made to control the design of the spillway, cascades, and stilling basin for the high dam now under construction at Vranov (Frain) on the Dyje River to serve the triple purpose of irrigation, flood control, and power production. The outlet works at the center of the dam control four discharge pipes 67 in. in diameter, by means of two Johnson and two spherical valves. The capacity of these controlled outlets is 6,070 cu ft per sec, whereas the maximum 50-year flood of 14,100 cu ft per sec is to be taken care of by nine overflow openings on the crest of the dam. Three of these discharge directly into the stilling basin; from the other six the water drops into the collecting cascades. A power house is to be located on the bank opposite the cascades.

Of particular interest is the tumble basin, in that horizontal eddies with vertical axes rather than the usual hydraulic jump are to be utilized to absorb the excess energy of flood waters. To this end, Prof. A. Smrček has designed two large baffle piers which have been constructed near the center of the channel directly in the path of the jets from the outlet valves. Each pier is 14 ft 9 in. high and presents a vertical face 21 ft wide to the discharging stream. The sketch in Fig. 2 illustrates the arrangement. The successful operation of these piers has already been proved by the high water of the 1933 season.

This structure is the first all-concrete dam to be built in Czechoslovakia, stone masonry being the more usual material in European practice. The dam is $180^1/_2$ ft high above the stream bed and has a crest length of 958 ft. Although designed as a gravity structure, it has a radius of curvature of 1,640 ft.

VRANOV DAM DURING CONSTRUCTION IN 1933, CZECHOSLOVAKIA'S FIRST CONCRETE DAM, NEAR BRNO

With the rapid development of water power that can be expected to follow the return of a productive economic cycle in Czechoslovakia, there can be no doubt that the well-equipped laboratory at Brno will continue to play an important part in the creation of safe and efficient structures and in the reclamation of many acres of farm lands now subjected to annual floods.

The National Hydrological Institute plainly forms an ideal medium for correlating all the activities of the government in the field of hydraulic construction. It is of course particularly adapted to the size of the state, in which the laboratories can be close both to the construction site and to the political center where the funds must be appropriated. It is noteworthy that erection of the model laboratory was readily accepted by the government as a necessary part of its river control program.

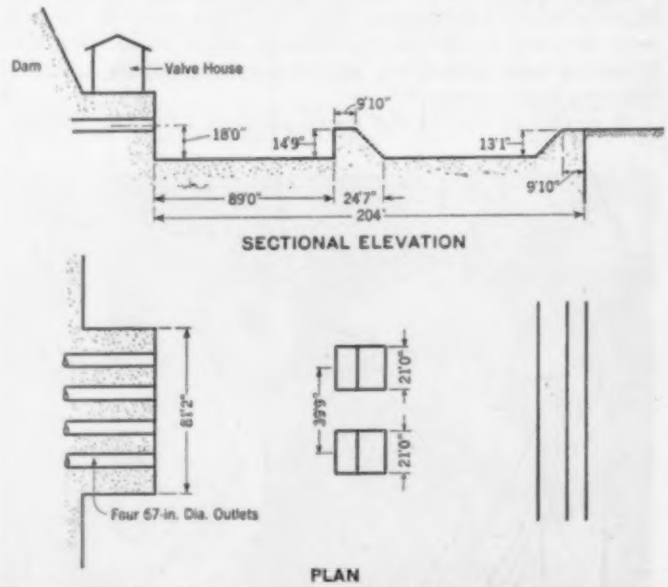
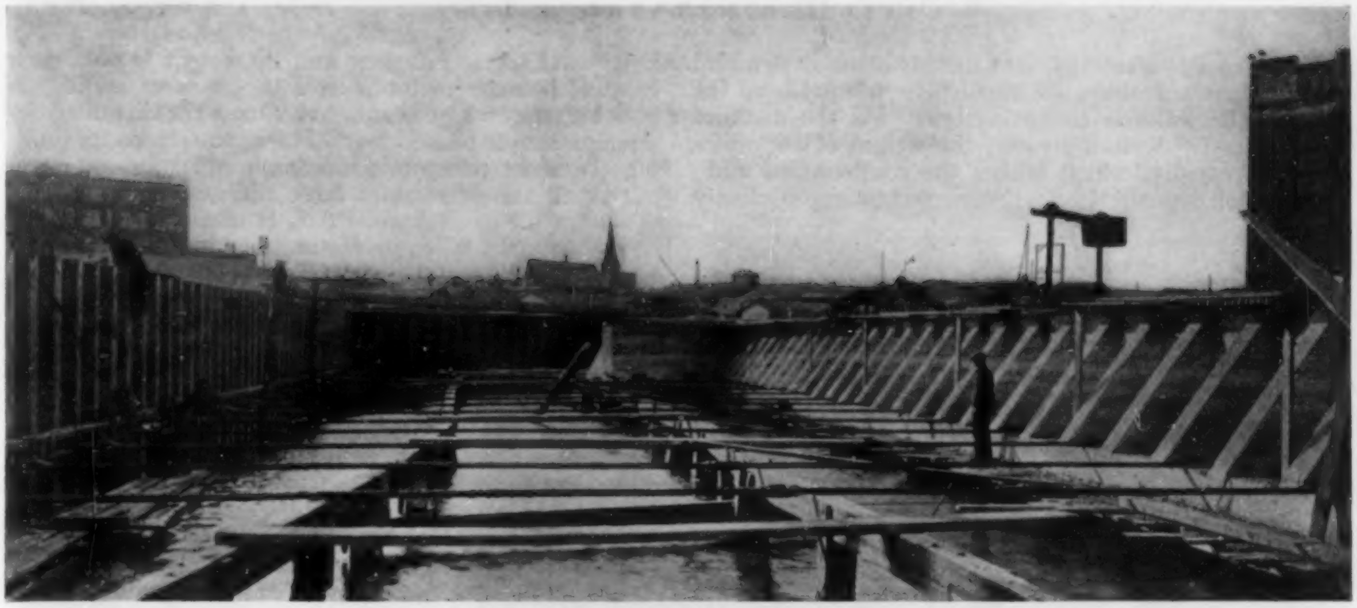


FIG. 2. BAFFLE PIERS AT VRANOV DAM DESIGNED TO DEFLECT DISCHARGING JETS AND CAUSE FORMATION OF ENERGY-ABSORBING EDDIES

Grateful acknowledgment is made to Dr.-Ing. Jan Smetana, Director of the T. G. Masaryk National Hydrotechnic Institute, for the photographs of models from that laboratory and for data concerning the work of the National Hydrological Institute; and to Prof. A. Smrček for photographs of, and information on the model of the Vranov Dam.



BEHIND A STEEL BULKHEAD BEFORE THE FILL WAS PLACED
Inside Wale, Anchor Rods, and a Well-Designed Anchorage

Details of Steel Sheet-Piling Bulkheads

Design and Construction of Tie Rods, Wales, Anchorages, Fender Timbers, and Bollards

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IN the November 1933 issue, Mr. Pennoyer presented briefly the essential theory involved in determining earth pressures and resulting loads on the various elements that make up a steel sheet-piling bulkhead. This article carries the subject a step further and describes methods of designing such details as tie rods, wales, and anchorages so that safe yet economical construction will result. Mr. Pennoyer points out a number of pitfalls which the designer of such bulkheads will wish to avoid and gives many practical erection hints that experience has proved will save cost.

AFTER the stresses on a bulkhead wall have been analyzed and the proper steel sheet-piling section has been selected, suitable tie rods, wales, and anchorage must be designed to prevent movement of the bulkhead wall in resisting the lateral loads of the confined earth and to hold it in line. If the bulkhead is for a wharf or slip, fender timbers are generally incorporated and very often a cap, sill, and mooring posts. In all such construction, the minimum thickness of exposed metal should at least equal the minimum thickness of the web or flange of the steel sheet piling, in order to ensure equal length of life under corrosion. The construction should be rugged and its factor of safety, if possible, should be at least as great as that of the bulkhead wall. The costs of field erection should receive careful consideration, and in this connection the maximum of simplicity is generally the maximum of economy.

DESIGN OF TIE RODS

If it is necessary to reduce the bending moment in the wall—between the wale at the top and the supporting soil at the bottom of the piling—to a minimum in order not to overstress the steel sheet piling, the elevation of the wales and tie rods should of course be chosen as low as possible consistent with economical erection. However, especially in tidal waters, the higher this elevation, the cheaper the erection, because the men can work longer between tides, and further, the load on the wales and tie rods is lightened, thus saving steel. The economy secured is still more marked if it is necessary to excavate for the anchorage. Therefore as a rule it is economical to choose an elevation as high as possible for the wale and to utilize the full safe strength of the steel sheet piling, in bending, between the wale and the bottom. It is well, however, when practical, to keep the top elevation of wales and tie rods within the zone of saturation of the soil to retard corrosion. The wale should never be more than 18 in. below water level, because below this level divers must be employed, and it is more economical to use heavier sheet piling or to reinforce the piling. Wales, even at mean low tide, are expensive to place because of the shortness of the intervals during which work is possible.

The load per foot of width on the bulkhead wall at the elevation of the wales and tie rods was developed in my previous article. The selection of the proper distance between rods is not difficult if it is kept in mind that the total load on the wall is the same, regardless of the number of rods, and therefore that the total amount of steel in the rods is the same, regardless of their spacing. Too short a center-to-center distance results in more rods than necessary in the bulkhead, which adds greatly to the cost of erecting them and constructing the anchorage. Further, the unit cost of rods of very small diameter is high. On the other hand, too great a distance between rods makes it necessary to use too large a rod, on which

the unit cost is also high, and increases the design and erection cost of the anchorage. By comparison, the steel in the wales is cheap material. As the distance between rods becomes greater, the weight of the wales increases rapidly, which makes the combination additionally uneconomical. If the tie rods are too closely spaced, channels for the wales light enough for efficiency would not be strong enough for ruggedness.

A 22-YEAR-OLD WHARF IN TROPICAL WATERS
Wales, Attached on the Outside, Are Badly Deteriorated

The center-to-center distance between the tie rods should equal an even number of widths of sheet piles. Usually the most economical distance is 6 or 8 piles, which for 16-in. piles means a length of 8 ft or 10 ft 8 in., although sometimes for very light bulkheads a distance of 13 ft 4 in. (10 piles) is advisable. As a rule the center-to-center distance between rods should be selected so that their diameter will lie within the most economical and practical range, which is from $1^1/_4$ to 3 in. When possible, $^1/_4$ in. should be added to the diameter to allow for corrosion, or else the rods should be wrapped in tarred burlap, wound spirally.

Upset rods, except in small sizes, offer a marked economy and are available for quick delivery. Standard diameters and lengths of upsets should be adhered to strictly, not only for economy in cost but also because standard upsets have from 15 to 30 per cent more metal at the root of the thread than in the remainder of the rod. The maximum corrosion usually occurs back of the nut, where the rods pass through the sheet pile, so this excess metal is very desirable. It is cheaper to put in additional upsets and turnbuckles in a rod than to specify rods longer than standard. Experienced contractors can drive an almost perfectly aligned wall and do not require in the rods much provision for adjustment. It has been a rule of thumb to incorporate two turnbuckles in rods over 45 ft in length.

DESIGN OF WALES

As illustrated in Fig. 1, a wale usually consists of two steel channels, bolted back to back with pipe separators. The size is selected from the tables of uniformly distributed loads in any handbook, the total load being the same as that on the tie rods. A wale thus selected is considerably stronger than necessary because the tie rods take the load directly from two sheet piles and because the wale is continuous over the tie rods except at the splices. This makes for added length of life and is desirable because of the difficulty of renewing the wale. For the same reason, in choosing between a shallower and a deeper channel, the shallower should be selected if its strength is sufficient and its weight is not much greater, because of its greater thickness of metal. If possible, the weight next heavier than that required for strength should be selected. There is no reason for limiting the sheet piling to a minimum thickness of metal unless all the accessories have the same thickness or more. This is cheap insurance, because the material is inexpensive and the wharf goes to pieces if the wales and tie rods fail through corrosion. Once they are in place and the fill is made, it is practically impossible to renew them, except at great cost.

Two types of steel channel wales are shown in Fig. 1, one attached on the outside or water side, and the other on the inside or fill side. The advantage of the outside wale is that its supporting action is positive. This type also requires a very small amount of field fitting and fabrication, with a corresponding saving in erection cost, and it eliminates the bolts and washers necessary for the attachment of an inside wale. On the other hand, its disadvantages are so marked that the inside wale is now almost universally used. Outside wales are exposed to corrosion from all sides and therefore have a much shorter life than the bulkhead itself, which is protected on one side by the fill. They form a catch-all for coal dust and other debris, which accelerate corrosion and constitute a projection on which ice can exert its full force. Such a wale also requires very heavy fender timbers in order that the gunwales of ships will clear the projecting ends of the tie rods. A photograph shows a 22-year-old wharf in tropical waters that clearly illustrates the deterioration of an outside wale long in advance of the bulkhead. For comparison, a bulkhead constructed with the wales inside is also shown as an illustration of a well-designed concrete anchorage. The trough of the upper channel is often filled with concrete for further protection. These bolts should have sufficient area of steel at the root of the thread to carry their proportionate share of the load on the tie rods, plus, if possible, an allowance of $^1/_8$ to $^1/_4$ in. in diameter for corrosion.

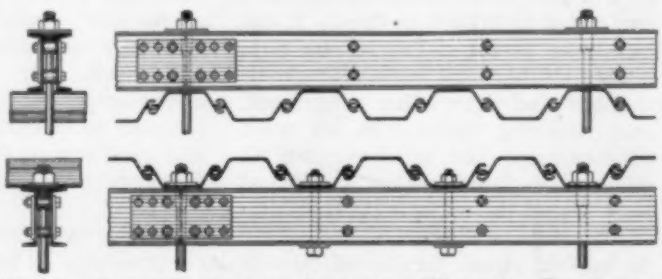

FIG. 1. TWO METHODS OF ATTACHING WALES TO BULKHEADS
Above; Attached on the Outside. Below; Attached on the Inside

HOLES FOR CONNECTIONS SHOULD NOT BE SHOP PUNCHED

Holes in the sheet piling for connections should be flame cut in the field after the piling is driven. The clearance in the interlocks and the difficulty of driving every sheet pile to exact grade renders irregularities unavoidable, and therefore shop punched holes for connections are generally useless. Flame cutting, in skillful

hands, is fully as satisfactory as drilling and generally is done at a great saving in cost. After driving, the sheet piling tops will often have to be flame cut to exact grade throughout the length of the wall.

A majority of steel bulkhead failures have been due to improperly designed anchorages. Many anchorages have been constructed without sufficient strength to hold the loads, or without provision for the transfer of stresses, or too close to the wall, and are glaring examples of the fact that "a chain is no stronger than its weakest link." The design should be studied just as carefully, with reference to conditions at the site, as that of the bulkhead wall, and if possible both should have the same factor of safety because if either fails the entire investment is lost.

The anchorage should be located far enough back of the bulkhead to be in stable soil. The angle of repose of the soil or fill, drawn from the intersection of the bottom of the channel and the bulkhead line to the tie-rod line, locates the plane of stable soil. If practical, the face of the anchor wall should be 10 or 15 ft back of this plane for additional safety.

Design of Anchorage Requires Consideration of Many Factors

Common sense, judgment, precedent, and experience, mixed with as much theory as possible, are necessary for the proper design of an anchorage. If the anchorage is to be placed in natural, compact, and undisturbed soil (not fill), the passive pressure or resistance of the soil can be used as a basis for its design, but if these conditions do not apply, the bearing and extraction value of the anchor piles must be utilized. A simple and practical anchorage is a continuous steel, wood, or reinforced concrete wall. The use of a continuous wall is illustrated in Fig. 2, in which the tie rod is assumed to pass through the center of the earth resistance, or in other words any

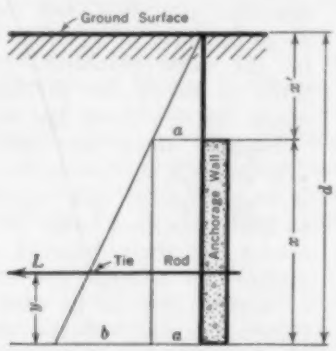

Fig. 2. An Anchorage Designed as a Continuous Wall
A Simple and Practical Type

tendency of the anchor wall to rotate is disregarded. All dimensions are in feet, and pressures are in pounds per square foot. In this figure the following notation is used:

p_p = increment of passive earth pressure

p_e = increment of active earth pressure

L = load on the wale, in pounds per linear foot

$a = x'(p_p - p_e)$

$b = x(p_p - p_e)$

The method of determining the value of p_p and p_e was described in my article, "Design of Steel Sheet-Piling Bulkheads," in the November 1933 issue. The net effective increment of pressure resisting movement of the anchor wall is $(p_p - p_e)$.

Assuming that the anchor wall is continuous throughout the length of the bulkhead and has sufficient strength

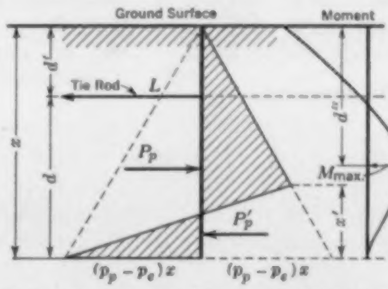

Fig. 3. Loads on Anchor Piles and Moments in Them
For Determining Size and Length of Piles

to act as a beam, then the area, as determined by the unknown height x, must be sufficient to develop enough passive resistance of the soil to prevent lateral movement due to the load on the wale, L, or

$$L = ax + \frac{bx}{2} = (p_p - p_e)\, x \left(x' + \frac{x}{2}\right)$$

The proper elevation of the bottom of the wall $(x' + x)$, can be judged within close limits. For wharves it will generally be found difficult to work in the dry deeper than 1 or 2 ft below low-water level, even after the steel bulkhead is driven.

$$\text{Let } d = (x' + x)$$

Solving this equation, the necessary height of the wall to resist the load on the bulkhead is found to be

$$x = d - \sqrt{d^2 - \frac{2L}{(p_p - p_e)}} \qquad [1]$$

It is well, of course, to add from 25 to 50 per cent, if practical, to the value of x for a factor of safety.

The tie rod should be located so as to pass through the center of gravity of the trapezoid representing the total earth resistance of the wall. Let y equal the distance between the tie rod and the bottom of the wall, in feet. Then

$$y = \frac{3x'x + x^2}{6x' + 3x} \qquad [2]$$

These same principles can be applied if the anchorage is not continuous but in separate sections at each tie rod.

If the anchorage is of vertical piles driven into the earth, into which the load of the tie rods is delivered near the tops of the piles and not through the center of soil resistance, as in the preceding case, there is a tendency for the piles to rotate. The piles must have a length sufficient to prevent this rotation and also sufficient section modulus as a cantilever. It is believed that the following method for the determination of the length and size of the piles, although not strictly accurate theoretically, is sufficiently so considering the inaccuracy of the basic assumptions as to earth loads.

The problem is shown graphically in Fig. 3, where the indicated heights are in feet and the following notation is used:

L = total load on tie rod, in pounds

w = effective width of the pile or piles at each tie rod bearing against the soil

$(p_p - p_a)$ = net effective increment of passive soil pressure, in pounds per square foot

Summing the horizontal forces

$$L - w(p_p - p_a)\left(\frac{x^2}{2}\right) + w(p_p - p_a)x . x' = 0$$

Summing the moments,

$$Ld - w(p_p - p_a)\left(\frac{x^3}{6}\right) + w(p_p - p_a)x\left(\frac{x'^2}{3}\right) = 0$$

By solving for x' in the first equation, substituting in the second, and simplifying, the following equation is obtained:

$$x^4 + \frac{4Lx^2}{w(p_p - p_a)} - \frac{12Ldx}{w(p_p - p_a)} - \frac{4L^2}{w(p_p - p_a)^2} = 0 \quad [3]$$

In this equation the only unknown is x, the necessary length of the pile or piles, which can be determined by trial and error. The location of the tie rod, d', is

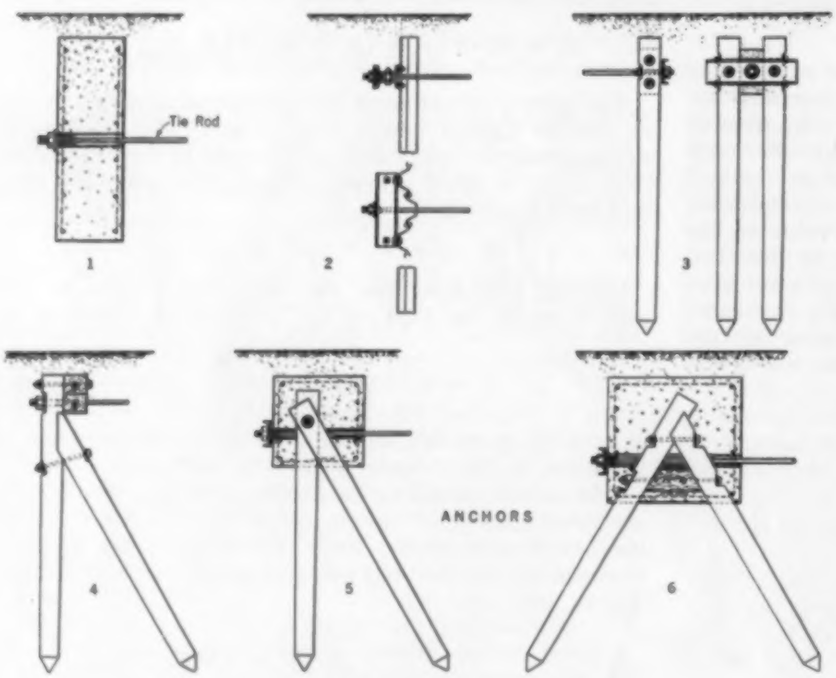

FIG. 4. COMMON TYPES OF ANCHORS FOR BULKHEADS
Advantages and Disadvantages of Each Type Discussed in Text

known, which gives the trial value of d in relation to x.

The bending moment on the pile or piles is determined approximately as follows:

$$M = L(d'' - d') - (p_p - p_a)\frac{d''^3}{6}$$

To find the location, d'', of the maximum moment and its value, differentiate as follows:

$$\frac{dM}{dd''} = L - (p_p - p_a)\frac{d''^2}{6} \quad d'' = \sqrt{\frac{2L}{p_p - p_a}}$$

Substituting,

$$M_{max} = L\left[\sqrt{\frac{8L}{9(p_p - p_a)}} - d'\right] \quad . \; . \; . \; . \quad [4]$$

the solution of which is in foot-pounds. The proper pile can then be selected or designed, based on the pile material, whether wood, steel, or concrete.

TYPES OF ANCHORS DISCUSSED

Some typical anchors subject to variations and combinations are given in Fig. 4. If the basis of design already given is used, types 1, 2, and 3 will be found to be unsafe, except in stable soil, because the bearing value of a fill is uncertain. Where loads are light and the fill is shallow, anchor types 2 and 3 can be designed to have enough section modulus, as well as cheapness and practicality, to be safe against rotation in the stable soil below the fill. These types of anchors can be built of steel sheet piling, steel H-beams, reinforced concrete, or wood. Steel sheet piling has the advantage of a large bearing area, but it will be found that the required section modulus will soon exceed the safe value as loads become heavier, unless a continuous wall is constructed. For medium loads, wide-flanged steel H-beams combine a large bearing area with high section modulus and give economy in cost as well as ease and certainty of driving. Reinforced concrete piles share these advantages but have the drawbacks of cost and difficulty of driving into hard strata.

Anchorage types 4, 5, and 6 (Fig. 4), utilize batter and vertical piles, whose bearing power and resistance to extraction respectively oppose the pull of the tie rods. The resistance of the anchor piles should be very carefully studied where the bulkhead is built in water and the piles will extend unsupported, except by fill, a considerable distance above the undisturbed, stable soil at the bottom. The bearing value of the piles is best determined by test piles, or if test piles are not practical, by the *Engineering News* formula, or by other formulas based on the same principle. The practical limit of batter for driving purposes is about 30 deg from the vertical.

Equally important are the vertical piles in resisting extraction. Information is fragmentary, and opinions differ as to the extraction value of piles with reference to their bearing value. Some authorities assume the extraction value of wood piles to be the same as their bearing value, but the writer's designs are based on an extraction value never more than 75 per cent of the bearing value of such piles. For concrete piles the writer uses 80 per cent of the bearing value, and for steel H-piles, not more than 60 per cent. Such values must be used with judgment and are subject to variation according to soil conditions. A decided advantage for steel is that it can be driven without damage into strata which neither wood nor concrete would penetrate, with correspondingly higher bearing and extraction values in some situations.

CONCRETE CAPS OFTEN FOUND MOST PRACTICAL FOR FRAMING ANCHORAGE AT TOP

All anchorages must be framed at the top to transmit the shear stresses from the rods to the piles. This is sometimes difficult to do with wood alone and for that reason concrete caps, such as shown in anchorages 5 and 6 (Fig. 4) are often found to be the most practical. They easily combine strength with cheapness of con-

struction, in that all field fitting is reduced to a minimum; irregularities in the driving and cutting of the piles are of no consequence, and the tops of the piles are preserved from rot or rust.

For steel sheet-piling wharves, fender timbers are advisable but not essential. They are an insurance against the ramming of the piling and the tearing of the interlocks by the bow of a vessel, and they also protect both the vessel and the piling from abrasion during docking, loading, and tide movement. Fender timbers can be designed to suit the ideas of the engineer and the particular conditions. Since continuous horizontal timbers, vertical timbers, or a combination of both can be bolted directly to the continuous steel wall, the construction is simpler and cheaper than for other types of wharves. An illustration shows a very rugged, practical, and popular type. All holes for connection bolts should be flame cut or drilled after the sheet piling is driven. Since in this type the bolt heads are buried in the fill and renewal of the timbers is therefore difficult, a renewable wearing strip 4 in. in thickness should be spiked to the timbers. All bolts, washers, and spikes in contact with wood should be galvanized.

Caps and sills can also be varied in design to suit the needs and ideas of the engineer and can be of concrete, wood, or steel, or a combination. Sometimes gantry-crane rails are supported by means of caps which distribute the concentrated loads directly to the top of the sheet piling. A very simple and economical cap is a steel channel laid with its trough on top of the piling. Holes are flame cut through the flanges, and piling and bolts are inserted. This cap has the advantage of being very stiff laterally, so that the bolts can be used to "pull" the tops of the sheet piles into a true line, and further it offers a means of connecting fender timbers or a sill to the bulkhead.

Mooring posts or bollards, when not formed of self-supporting pile clusters, should have a foundation of concrete, and if placed adjacent to the steel bulkhead should be attached rigidly to the steel. When mooring posts place additional stresses on the bulkhead, these should be taken care of by an additional or a heavier tie rod, and the anchorage should be strengthened where each post is connected. A 3-in. hemp hawser is considered able to exert a pull up to 60,000 lb, which in itself requires a tie rod $2^1/_8$ in. in diameter upset to $2^5/_8$ in.

PLACING THE FILL

The fill behind a bulkhead cannot be placed too carefully. It is at this time that the stresses on the bulkhead and anchorage are at a maximum and the structure is generally least able to resist loads. After the fill is completed, the lateral pressures of the earth decrease very markedly because of the settlement of the fill and its consequent steeper angle of repose. The operation of backfilling should be closely supervised by an engineer who is capable of recognizing the stresses being set up and of directing the procedure.

The fill should never be placed towards the bulkhead, because such a method usually pushes a mud wave toward the piling, which greatly increases the lateral load on it when the mud wave is finally confined. The safest method is to fill, in layers, the whole distance between the anchorage and the bulkhead, keeping the surface of the fill as nearly level as possible. By this means the mud wave is eliminated, and the anchorage increases in stability as the load increases on the bulkhead.

Very often the fill is dredged from the channel in front of the bulkhead, but in this case the resulting load, especially in the case of hydraulic fills, is very heavy, and the bulkhead should be designed for it. Clay is the very best material to place against the steel because it is not acid, excludes oxygen, and also prevents the percolation of impure surface water—all of which reduces corrosion to a minimum. A few feet of thickness is all that is needed of such material. Back of this any good fill material can be used. Coal ashes should never be placed against steel.

FENDER TIMBERS ON A STEEL SHEET-PILE WHARF

It is better to order the sheet piling safely long than too short. If the toe starts to move out, the whole structure soon goes. Borings should go deep enough to make certain that there are no unstable strata below the stable one. The piling should be driven through any doubtful soil and deep enough into good sand, gravel, clay, or to rock. The fill behind the bulkhead places such a heavy burden on any unstable stratum that it may cause the entire mass, including the structure above, to slide.

On the other hand, the cheapest possible job is sometimes wanted in very shallow water when it is obvious that a deeper channel will pay for itself or will be needed very soon. Obviously, the whole investment in the wharf for shallow water will be wasted if the sheet piling is ordered only long enough for present needs. It is surprising how difficult it is sometimes to demonstrate the economy of buying the sheet piling long enough to provide for the deeper channel when needed.

A very important consideration is that of financing the construction. Civic organizations or promoters, contemplating wharf facilities, always have dreams of the future, and generally their imagination pictures their port as so important that ocean liners must be accommodated. Such instructions as these appear frequently: "Design for a 35-ft channel [sometimes a 40-ft] but only a 25-ft channel will be required at present." Others state that railroad tracks, gantry cranes, or warehouses will be incorporated later. Very few who request such designs realize how much every foot of added depth, every pound of surcharge, and every added foot of freeboard increases the cost. Generally the design and investment should be made for present requirements, but it should be certain that the investment can all be used when the time comes for enlargement. In some cases it is an impossibility for future plans ever to materialize, and the smaller investment, for the present need, may meet the permanent need. The difference in cost might be the difference between a profitable and a losing venture. Further, obsolescence usually governs the life of a structure, but if greater needs actually develop, the interest on the saving in the meantime might pay for future additions.

ACKNOWLEDGMENT

Acknowledgment is made of the assistance of H. Blum in developing the method of calculating the length of anchor piles and the bending moment in them.

Problems in Locomotive Acceleration

Formulas for Tractive Power and for Time and Distance Required to Attain Speed

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A GENERALLY adopted method of determining the distance required for a given locomotive and train to attain a given speed on a given grade is one requiring a laboriously calculated table of locomotive tractive efforts. In this article Professor Barrow attacks the problem by the use of curves and of equations developed to fit them closely. Although cubic equations are involved, usable formulas are derived and short and direct methods of applying them are presented, which may be helpful in solving problems of speed and time of acceleration of trains and the distance required.

Courtesy Pennsylvania Railroad

Crack Passenger Train Which Travels at an Average Speed of 55 Miles per Hr and Frequently Reaches That of 80 Miles per Hr on the Straightaway

The tractive power of a steam locomotive depends on and may be limited by: (1) the capacity of the boiler to make steam; (2) the capacity of the mechanism to convert the steam power into motion, which capacity is dependent on the diameter of the cylinders and driving wheels; and (3) the adhesion between the rails and the drivers, which in turn is dependent on the weight of the locomotive on its drivers. At low speed the limiting factor is likely to be rail adhesion, which is from one-fifth to one-fourth the weight on the drivers. At high speed the steam-making capacity of the boiler may be the limiting factor. Locomotives exert their greatest power at starting. The power then decreases slightly until the critical speed is reached, after which it drops rapidly, since full boiler pressure cannot thereafter be maintained in the cylinders at full cut-off. The exact relationship between power and speed can be determined by dynamometer car readings, by tests at a testing plant, or by computation.

In Fig. 1 are shown curves of the relation obtained by generally accepted methods between speed and boiler tractive power of a simple 12-wheeled locomotive with the characteristics indicated. At speeds above the critical, the relationship may be expressed closely by:

$$T = \frac{C}{S} + k$$

or

$$ST = C + Sk \quad . \; . \; . \; . \; . \; . \; . \; [1]$$

in which T is the tractive effort, S is the speed, and C and k are constants. An empirical formula for the locomotive in Fig. 1 may be obtained by the following method. From the curve of gross tractive power, tabulate values of T for corresponding values of S, as shown in Table I. By taking the tabulated data in groups of three speeds, as shown, two equations are obtained:

Table I. Data and Calculations for Gross Tractive Effort For Locomotive in Fig. 1

	S	T	ST	T_e	Difference
	10	22.10	221.0	24.0	+1.90
	15	17.40	261.0	17.15	−0.25
	20	14.50	290.0	13.72	−0.78
Total . .	45		772.0		
	25	12.2	305.0	11.69	−0.51
	30	10.5	315.0	10.31	−0.19
	35	8.8	308.0	9.34	+0.54
Total . .	90		928.0		

$$772 = 3C + 45k$$
$$928 = 3C + 90k$$

By subtracting, $156 = 45k$

or $k = 3.47$

and $C = 205.3$

By substituting these values in the general Equation 1, the equation for this particular locomotive is found, as follows:

$$T = \frac{205.3}{S} + 3.47 \quad . \; . \; . \; . \; . \; . \; [2]$$

Its degree of accuracy can be seen in Table I, by comparing the values of T, obtained from the curve, with those of T_e, computed by Equation 2. Differences are shown in the final column. It is seen that the computed values agree closely with the observed values with the exception of the first. Near the critical speed, the curve is changing rapidly in direction, and it cannot be expected that the equation will be as accurate near that speed as at higher speeds. The constants C and k may be evaluated even more closely by the method of least squares.

On level, straight track in still air, train resistance, R,

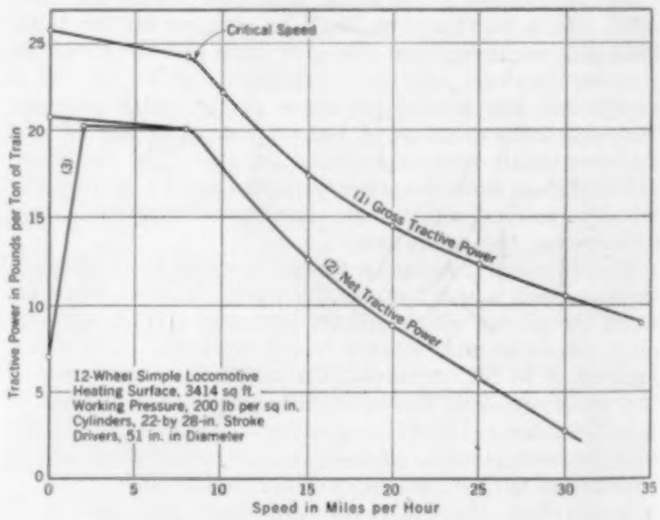

Fig. 1. Diagram of Boiler Tractive Power at Rim of Drivers of a 12-Wheel Simple Locomotive

includes friction on journal bearings, rolling resistance of wheels on rails, resistance due to oscillation and concussion, and atmospheric resistance. No simple relationship between this combination of factors and the speed of the train can be expressed, but numerous equations have been proposed and published. One such formula is:

$$R = 3.5 + 0.0055\, S^2 \quad . \; . \; . \; . \; . \; . \quad [3]$$

Grade resistance very nearly equals 20 lb per ton for each 1 per cent of grade. It is obvious that there is a speed on a given grade at which the tractive effort of the locomotive will just equal the resistance of the train plus that of the locomotive. By observation, the locomotive characterized in Fig. 1 will make 8.08 miles per hr on a 1 per cent grade with a certain train load. To find the speed which the same train will make on a 0.5 per cent grade, equate the total resistance of the train and grade to the power of the locomotive, remembering that the tractive effort of the locomotive is in pounds per ton and that grade resistance must be deducted. Then

$$\frac{205.3}{S} + 3.47 - (0.5 \times 20) = 3.5 + 0.0055\, S^2 \quad . \; . \quad [4]$$

Two methods that are much shorter than Strurm's general method or even Cardan's method, may be used for the solution of cubic equations. Of these, one is graphical and the other requires a table of hyperbolic functions. By transformation, Equation 4 becomes

$$0.0055\, S^3 + 10.03\, S - 205.3 = 0 \quad . \; . \; . \; . \; . \; . \quad [5]$$

or

$$S^3 + 1{,}828\, S = 37{,}700 \quad . \; . \; . \; . \; . \quad [6]$$

By the graphic method, let

$$0.0055\, S^3 + 10.03\, S - 205.3 = y$$

Assume a reasonable value of S and solve for y. On cross-section paper and at a convenient scale, plot S on the x-axis against the calculated value of y. Assume another value of S; again solve for y and plot, bearing in mind the algebraic sign of y. If the points are not too far from the x-axis, a line joining these points, produced if necessary, will cross the x-axis at a point indicating the maximum speed sought. For the example here considered, this procedure gives 17.6 miles per hr as the maximum speed attainable with the given load on a 0.5 per cent grade.

To solve with the table of hyperbolic functions, Equation 6 should be used in the general form,

$$S^3 \pm 3bS = 2c$$

in which b and c are constants. When b is positive,

$$S = 2b^{0.5} \sinh \left(\frac{1}{3} \sinh^{-1} \frac{c}{b^{1.5}} \right)$$

When b is negative, $S = 2b^{0.5} \cosh \left(\frac{1}{3} \cosh^{-1} \frac{c}{b^{1.5}} \right)$.

In the problem under consideration,

$b = 601$, $b^{0.5} = 24.5$, $c = 18{,}606$, $b^{1.5} = 14{,}706$.

Hence S equals 17.5 miles per hr, which agrees closely with the result obtained by the graphic method.

To determine the time and distance required for a locomotive to accelerate, an equation is needed to give the tractive power of the locomotive that remains after train, grade, and curve resistance have been overcome. This equation is derived from the curve of net tractive power in the same way that Equation 2 is derived from the curve of gross tractive power, and has the form of Equation 1.

In Fig. 1 the upper curve gives the gross tractive power and the lower gives the net tractive power. It is generally assumed that, on starting from rest, train resistance is about 20 lb per ton of train, which decreases very rapidly up to a speed of 2 or 3 miles per hr and then decreases more slowly until the critical speed is reached. This is illustrated in Fig. 1 by the part of the lower curve marked "3." However, in dealing with problems of acceleration, it is assumed by some authorities that at speeds from zero to 3 or 4 miles per hr the train resistance is equal to that of the same train when slowing down to these speeds. This is illustrated in the left-hand part of Curves 1 and 2.

If it is desired to determine the time and distance needed to accelerate from zero to some speed above the critical, the solution must be reached in two steps, one from zero to the critical speed, and the other from the critical speed to that under consideration. Assuming that the tractive power of a locomotive decreases uniformly from zero to the critical speed, the mean value is used in the familiar equations,

$$L \text{ (in feet)} = 70 \frac{W}{T} (S_2^2 - S_1^2)$$

and

$$t \text{ (in seconds)} = \frac{15}{11} \left(\frac{L}{S_1 + S_2} \right)$$

in which W is the weight of the train in tons, and t is time.

Above the critical speed, the following method is applicable. Transform the fundamental equation, $F = Ma$.

$$\text{Then} \quad a = \frac{F}{M} = \frac{F}{\dfrac{W}{32.2}} = \frac{32.2\, F}{W}$$

in which F equals force, M equals mass, a equals acceleration, and W equals weight. Expressing weight in pounds per ton and taking acceleration as 21.95 miles per hr instead of 32.2 ft per sec, the equation becomes:

$$a = \frac{21.95\, F}{2{,}000} \quad . \; . \; . \; . \; . \; . \; . \; . \quad [7]$$

Substituting $\frac{dv}{dt}$ for a; and the tractive power of the locomotive, $\frac{C + Sk}{S}$, from Equation 1 for F,

$$\frac{dv}{dt} = \frac{21.95}{2{,}000} \left(\frac{C + Sk}{S} \right) \quad . \; . \; . \; . \; . \; . \quad [8]$$

from which

$$dt = \frac{2{,}000}{21.95} \left(\frac{S}{C + Sk} \right) dv \quad . \; . \; . \; . \quad [9]$$

and

t (in seconds) =

$$\frac{91.11}{k^2} \Big[C + Sk - C \log_e (C + Sk) \Big]_{v_1}^{v_2} \quad . \; . \; . \quad [10]$$

As this equation always will be integrated between the limits of two definite speeds, the constant of integration need not be considered. If it is desired to express Equation 10 in feet per second instead of miles per hour the constant 62.2 must be substituted for 91.11.

To obtain the distance, L, required to accelerate to the given speed, substitute in the equation, $dL = vdt$, the value of dt in Equation 9, or

$$dL = 91.11 \frac{v^2 dv}{C + Sk} \quad \ldots\ldots [11]$$

$$\text{Then } L \text{ (in feet)} = \frac{91.11}{k^3}\left[\frac{(C + Sk)^2}{2} - 2C(C + Sk) + C^2 \log_e (C + Sk)\right]_{v_1}^{v_2} \quad \ldots\ldots [12]$$

An equation of the general form is then derived, in which a, b, c, and d are constants:

$$T = \frac{S + c}{a + bS + dS^2} \quad \ldots\ldots [13]$$

This equation will fit the curve of net tractive effort of a simple locomotive except near the critical speed, where it rises slightly above the true curve. The constant c in the numerator and the constant a in the denominator are solely for the purpose of giving the net tractive effort when S is zero. Omitting these, Equation 13 becomes

$$T = \frac{1}{b + dS^2} \quad \ldots\ldots [14]$$

or

$$b + dS^2 = \frac{1}{T}$$

The numerical constants in this equation may be found for the locomotive in Fig. 1 by the method used in making up Table I.

From Equation 14, expressions for time and distance can be derived:

$$t = 30.34\ (3bS + dS^3) \quad \ldots [15]$$

$$L = 7.59\ (6bS^2 + dS^4) \quad \ldots [16]$$

By this method the numerical values of the constants were found to be: for b, 0.037, and for d, 0.000087, and Equation 14 becomes

$$T = \frac{1}{0.037 + 0.000087\ S^2} \quad \ldots [17]$$

which is the equation for this particular locomotive. Table II shows how closely the computed values correspond to the observed values.

In order to fit the curves of tractive power more accurately to some locomotives, equations in the following form should be used:

$$T = \frac{S}{a + bS + dS^2}$$

or

$$\frac{S}{T} = a + bS + dS^2 \quad \ldots\ldots [18]$$

From these, simple expressions for time and distance of acceleration result. The equation for time thus becomes

$$t = 91.11 \left(a \log_e S + bS + \frac{dS^3}{3}\right) \quad \ldots [19]$$

and that for distance

$$L = 91.11 \left[a\left(\frac{S^2}{2} \log S - \frac{S^4}{4}\right) + \frac{bS^2}{2} + \frac{dS^4}{12}\right] \quad . [20]$$

The numerical values of the constants in Equation 18, to fit the curve of net tractive power in Fig. 1, are found in a manner similar to that used in preparing Table I.

By grouping the data in Table III, as shown, three equations are obtained from which the constants may be calculated as before, a being 0.75; b, −0.113; and d, 0.00945. The equation for this locomotive then is

TABLE II. COMPUTED VALUES AND OBSERVED VALUES FOR NET TRACTIVE EFFORT—LOCOMOTIVE IN FIG. 1

S		T	T_c	DIFFERENCE
Ft per sec	Miles per hr			
14.7	10	17.9	17.9	0.0
17.6	12	15.6	15.6	0.0
20.6	14	13.6	13.5	0.0
23.5	16	11.8	11.8	0.0
26.4	18	10.2	10.2	0.0
29.4	20	8.8	8.9	0.1

$$T = \frac{S}{0.75 - 0.113\ S + 0.00945\ S^2} \quad . [21]$$

Equation 21 gives values of T at critical speed more closely than Equation 2, and when used between speeds of zero and 20 miles per hr gives results very close to the values computed by means of approved tables, with variations on the safe side. Between speeds of 20 and 30 miles per hr the curve of net tractive power of this locomotive (Fig. 1) is seen to be a straight line, and this is true for a number of simple locomotives for which I have calculated formulas.

TABLE III. DATA AND CALCULATIONS FOR NET TRACTIVE POWER For Locomotive in Fig. 1

	S	T	S/T	S^2	T_c	DIFFERENCE
	10	17.9	0.558	100	17.6	−0.3
	12	15.6	0.768	144	16.2	+0.6
Total . . .	22		1.326	244		
	14	13.5	1.038	196	13.2	−0.3
	16	11.8	1.345	256	11.7	−0.1
Total . . .	30		2.383	452		
	18	10.2	1.765	324	10.0	−0.2
	20	8.8	2.270	400	8.7	−0.1
Total . . .	38		4.035	724		

It should be noted that the equation for a curve of rapidly changing radius cannot be made to fit a straight line, and it is recommended that in dealing with these problems the curves of gross and net tractive effort of the locomotive be plotted in order to determine the limits of the equations. For one four-cylinder compound locomotive investigated, it was found that the curve of gross tractive effort was a straight line from the critical speed to about 20 miles per hr, after which it curved downward. Equations 15 and 16 also follow closely the curve of tractive power at speeds below the critical and give the tractive effort at starting for a locomotive with a "booster."

Equation 1 has the advantage that it may be derived from the curve of power for a locomotive on a level track and applied to the same locomotive on a grade by combining the grade resistance with the constant k. However, Equation 14 must be computed anew for each grade unless a constant is inserted, giving the equation the following form:

$$T = \frac{1}{b + dS^2} - k \quad \ldots\ldots [22]$$

This form may also be used to calculate t and L, but I have integrated these expressions and find that they are too complicated for practical use.

Fourth Street Viaduct, Los Angeles, Completed in 1931
Ribs of River Span Designed as Three-Hinged Arches, with Temporary Concrete Hinges Which Were Later Concreted

Temporary Hinges in an Arched Bridge

Their Adoption for Fourth Street Viaduct, Los Angeles, Proves Economical

By Leon Blog

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With no American precedent to guide them, the designers of the Fourth Street Viaduct, Los Angeles, successfully planned and erected a reinforced concrete arch over the Los Angeles River by making use of temporary hinges. This $2,800,000 structure, which crosses fifty railroad tracks and three streets, presented many difficulties, not the least of which was the design and erection of the four arch ribs in the 254-ft river span. The hinges, three for each rib, consisted of short reinforced concrete columns located on the axis of the arch, and containing 3 per cent of spiral steel. After the centering was struck, the crown hinge was concreted in, but the hinges at the abutments were not closed until the superstructure was completed. Secondary stresses due to rib shortening, concrete shrinkage, and abutment movement were thereby reduced, with the result that there was a considerable saving of material in the ribs. During construction careful measurements were made of the behavior of ribs, hinges, and abutments for the purpose of adding to engineering knowledge of the action of this unusual type of construction.

In the program of bridge construction of Los Angeles, the largest all-reinforced-concrete project was that of replacing an old wooden viaduct crossing the Los Angeles River at Fourth Street, completed in 1931. The over-all length of the new viaduct is 2,703 ft, consisting of approaches, 10 abutments, and 18 piers. Seven of the arches are from $64^1/_2$ to $117^1/_2$ ft in span, and the central span across the channel of the river has a length of 254 ft and a rise of 33 ft. The remaining parts of the viaduct are of slab and girder construction. The bridge carries two street railway tracks on its 56-ft roadway and provides for two sidewalks. Its over-all width is 71 ft.

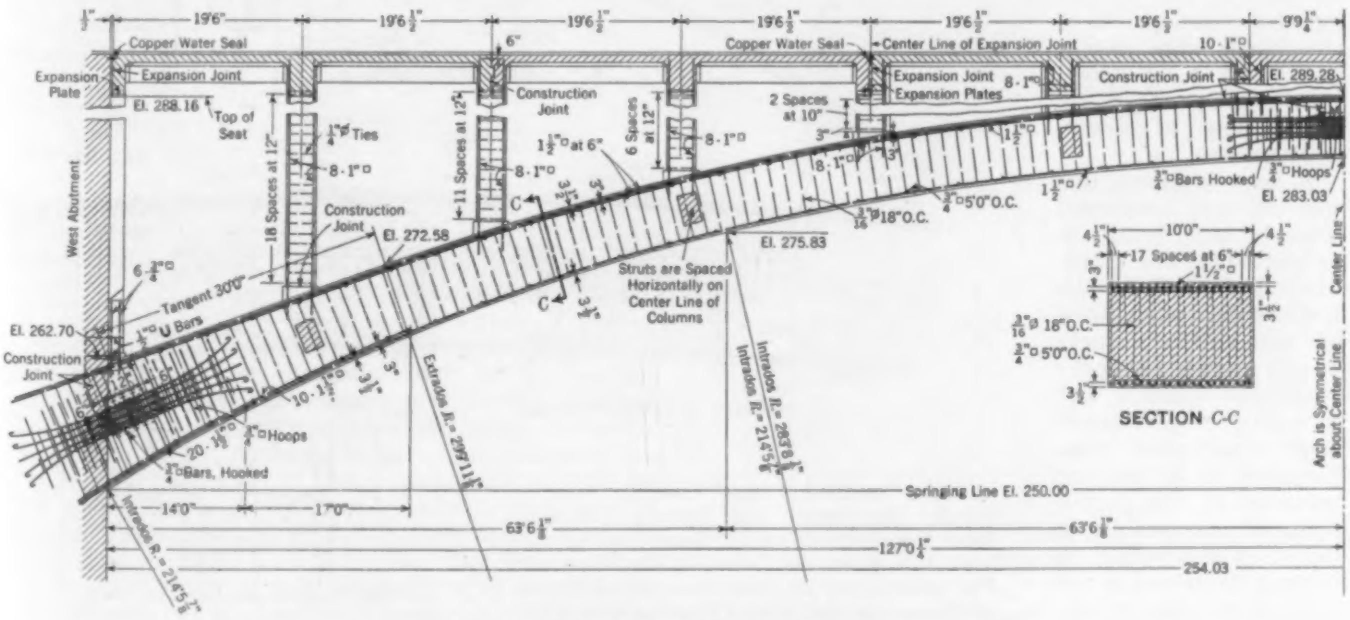
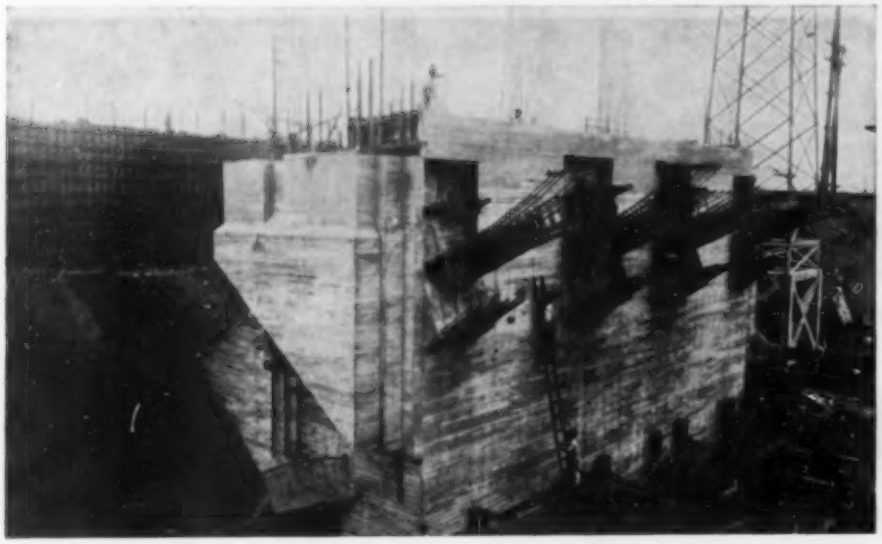


Fig. 1. Half Elevation of Railway Rib
Three-Hinged Arch of Reinforced Concrete, Fourth Street Viaduct

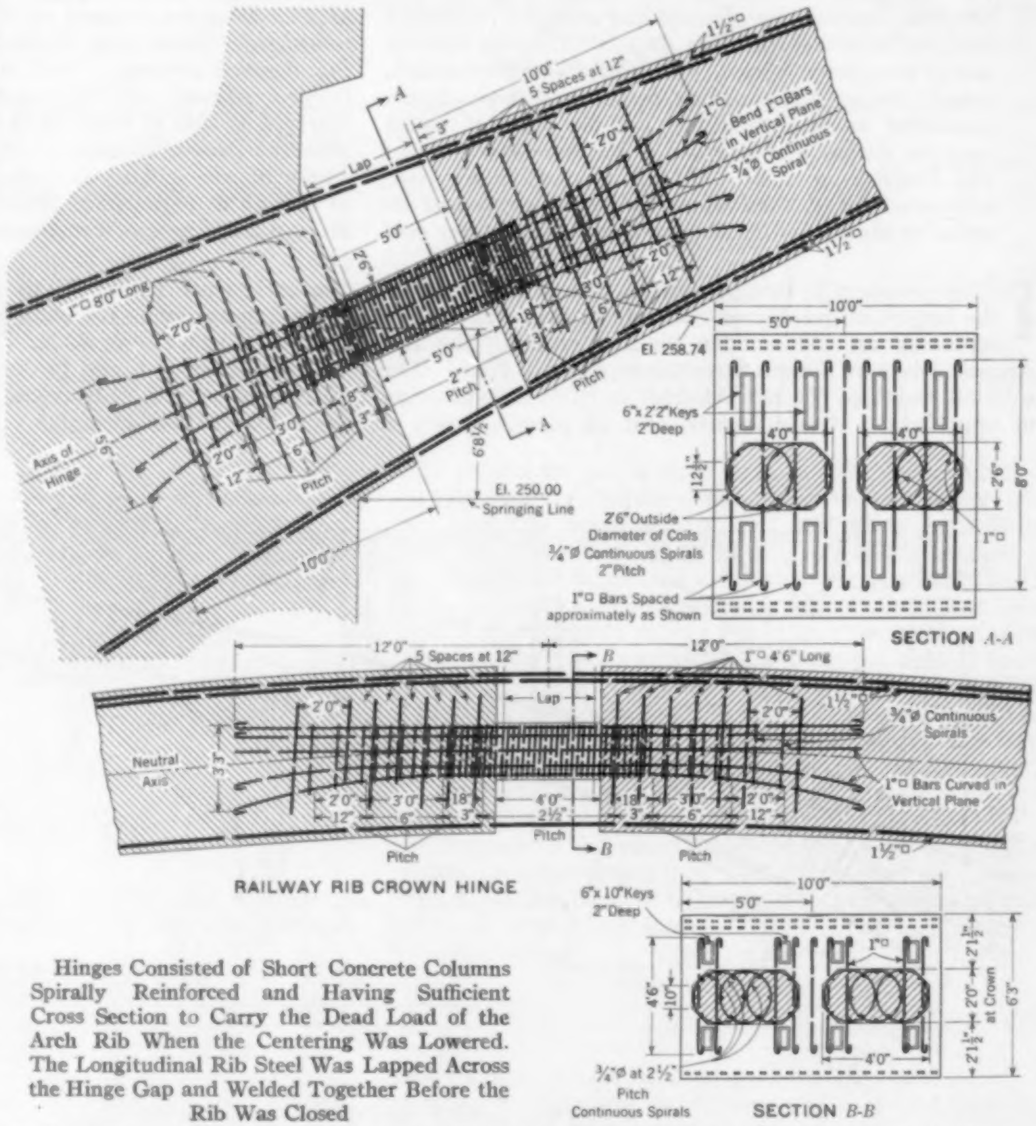


ABUTMENT FOR ARCH RIBS WITH REINFORCING STEEL OF TEMPORARY HINGES IN PLACE

Of principal interest is the river span, of the open spandrel type, in which the loads are carried on four ribs. The two outside ribs carry the sidewalk and highway loads, and the two inner ones carry the street railway and part of the highway load. To reduce stresses due to rib shortening, shrinkage, and abutment movement, the ribs were designed to act as three-hinged until the completion of the superstructure. The temporary hinges were short concrete columns reinforced with 1 per cent longitudinal and 3 per cent spiral steel and were centered on the axis of the arch at the springing and crown points of the ribs. The design was based on an allowable stress of 2,500 lb per sq in. of hinge, or column, cross section.

So far as is known, this was the first reinforced concrete arch bridge to be erected by the three-hinged method in the United States. Much has been written vaguely concerning stresses caused by rotation and spreading of abutments with considerable disagreement as to the importance of this factor, somewhat more about the effect of shrinkage on stress in structures, something about plastic flow, and little about the behavior of such hinges as were to be used. As for the effects of deflections to be expected in such a long three-hinged arch, no data were available. The largest open spandrel arch which Los Angeles had constructed up to that time was of 215-ft span and 40-ft rise. The total measured deflection of that arch was 0.165 ft, of which 0.043 ft was calculated to have been due to dead-load rib shortening, and the remaining 0.122 ft primarily to shrinkage. This was a hingeless arch.

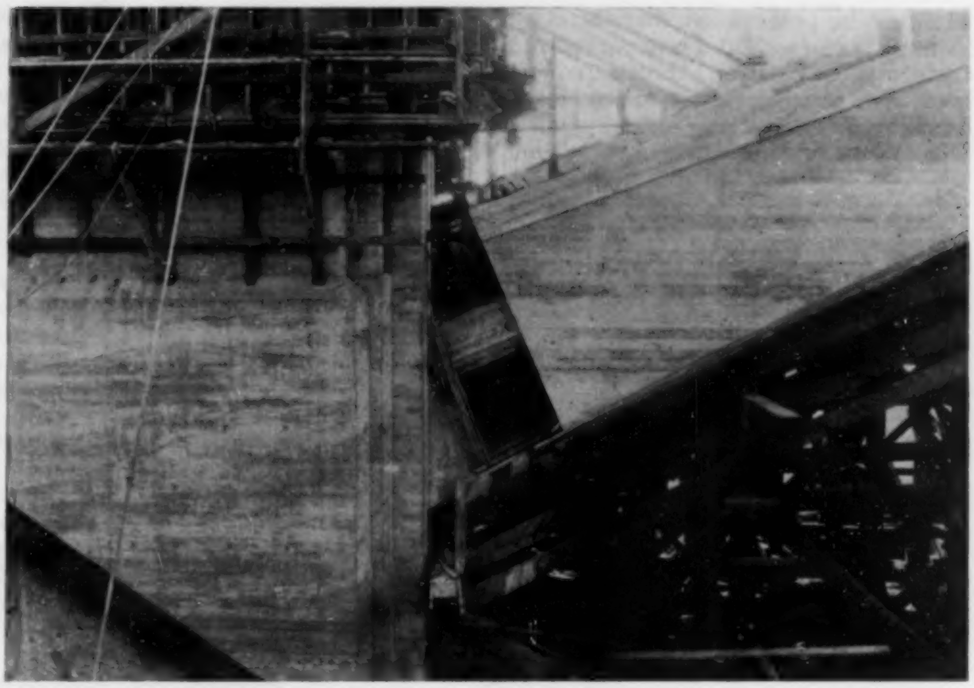
Probable deflections at various construction stages of the Fourth Street arch were analyzed by means of a model, using the Beggs' method. Based on our previous experience, the stresses indicated by this analysis were so large that serious distortion of the arch axis would occur were they correct. Then, too, it was probable that on account of the great thickness of the hinges they would not, in fact, afford true pin action, which is a prerequisite for the assumption of zero moment at the hinges in the analysis. If the hinges



Hinges Consisted of Short Concrete Columns Spirally Reinforced and Having Sufficient Cross Section to Carry the Dead Load of the Arch Rib When the Centering Was Lowered. The Longitudinal Rib Steel Was Lapped Across the Hinge Gap and Welded Together Before the Rib Was Closed

FIG. 2. DETAILS OF TEMPORARY CONCRETE HINGES AT SPRINGING LINE AND CROWN OF ARCH RIBS

did not afford pin action, the line of thrust would be eccentric to the theoretical location of the hinges on the neutral axis of the rib, and unallowed for and possibly large bending stresses would occur in the hinges. To settle some of these questions, and to aid in the design of future arches of this type, arrangements were made to obtain a series of field measurements.



Arch Rib and Temporary Abutment Hinge Poured

Since all the ribs were similar in design and construction, and since similar observations were made on all, only the railway ribs will be discussed, as they involve larger values and are as instructive as the highway ribs. The clear span of each rib is 254.03 ft; the intrados rise at the crown is 33.03 ft; and that at the quarter point, 25.83 ft. Each rib is 10.00 ft wide, and has a vertical thickness at the crown of 6.25 ft, and at the springing line, of 12.70 ft. In Fig. 1 are shown the details of the adopted design and in Fig. 2 the details of the temporary crown and abutment hinges. A comparison of the calculated sizes, stresses, and reinforcing steel, both with and without the temporary hinges, is given in Table I. It is evident that the total stresses for the hinged rib, even though reduced in cross section, were smaller. The estimated saving due to the use of the temporary hinges was $19,000.

Table I. Two Designs of Railway Rib Compared

Item	Hingeless		With Temporary Hinges	
	Crown	Springing Line	Crown	Springing Line
Width of rib, in feet	12.5	12.5	10.00	10.00
Depth of rib, normal to axis	6.5	13.0	6.25	11.00
Extrados reinforcing, in square inches	120	182	45	68
Intrados reinforcing, in square inches	120	182	45	68
Dead load stresses, in pounds per square inch	330	205	377	263
Live load stresses, in pounds per square inch	160	151	294	285
Temperature stress for a drop of 35 F, in pounds per square inch	204	274	150	291
Rib shortening stress, in pounds per square inch	168	240	0	0
Total stress, in pounds per square inch	862	870	830	839

Design of Hinges

This type of hinge has been used in the design of European bridges but with this difference: the longitudinal extrados and intrados reinforcing were made continuous through the space above and below the hinge, whereas in the Fourth Street viaduct the bars were discontinuous but lapped horizontally. They were welded together just before the hinges were concreted in. Observations on the lapped, unwelded bars proved that there was ground for the belief that if the bars had been continuous they would not have flexed readily and therefore the pure hinge action assumed in the design would not have been realized. Consequently the desired reduction of secondary stresses would not have been secured. The method of making the laps and welds is illustrated in one of the accompanying photographs.

In accordance with the practice of Scott and Consideré, the length of the columns or posts which formed the temporary hinges was limited to twice their least dimension; that is, they were true short columns. Under a direct central load such as they would receive in a true three-hinged arch, they were not expected to bend, and no provision against bending was made except that the high unit stress of 2,500 lb per sq in. was allowed. From experiment it is known that a spiral column has great ductility and will carry a greater ultimate load than will an ordinary tied column with only longitudinal reinforcement.

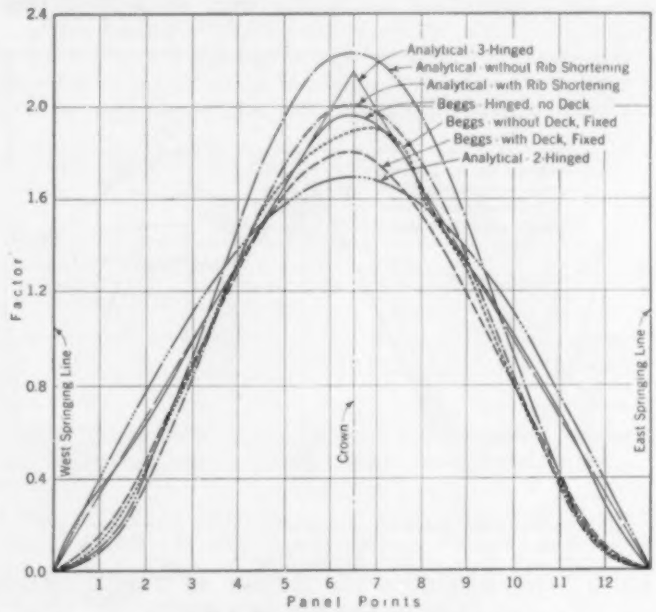


Fig. 3. Influence Lines for Horizontal Thrust in Railway Arch Rib

In Figs. 3 and 4 the mathematical analysis of the rib action is compared with the results of model studies.

It should be pointed out that there is no study for the moment of the arch in the two-hinged condition. In the mathematical analysis, thrust and moment for dead load were reduced for rib shortening. Model analysis automatically takes rib shortening into account, and the terms "with rib shortening" and "without rib shortening"

ARCH RIBS IN THREE-HINGED STATE WITH CENTERING LOWERED

do not apply to the Beggs curves. The effect of the hinges and of the operation of the deck on the value of the moment at the springing line was considerable.

TABLE II. OBSERVED LENGTHENING OF THEORETICAL ARCH SPAN AND ROTATION AND SETTLEMENT OF ABUTMENTS

Stage	Time	Lengthening of Arch Span In Feet	Rotation in Sec of Angle Abutment 3	Rotation in Sec of Angle Abutment 4	Settlement in Feet Abutment 3	Settlement in Feet Abutment 4
1	Immediately before decentering	0.02*	69†	16†	...	...
2	Immediately after decentering	0.048	53‡	78‡	...	...
3	Immediately after crown hinge closure	0.089	91‡	86‡	...	...
4	Immediately after spring hinge closure	0.104	101‡	140‡	...	...
5	Ten months after decentering	0.102	101‡	140‡	0.036	0.060

* shortening † inward rotation ‡ outward rotation

After the centering was lowered, the crown extrados and intrados longitudinal reinforcing were welded together

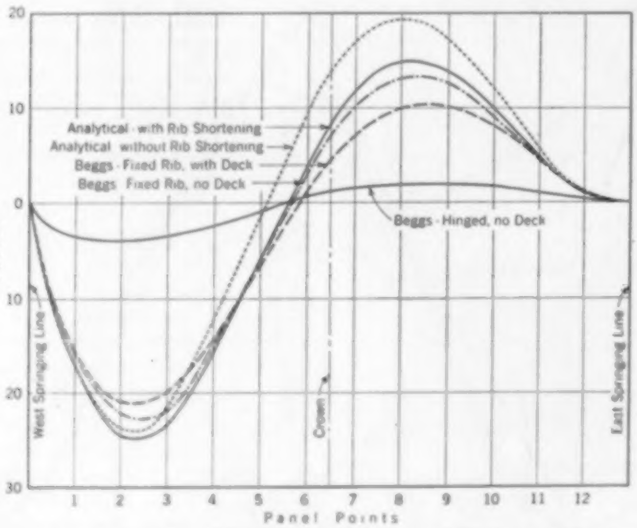


FIG. 4. INFLUENCE LINES FOR MOMENT AT WEST SPRINGING LINE OF RAILWAY ARCH RIB

and the hinge concreted in, thereby converting the rib from a three- to a two-hinged one. The reasons for doing this were: (1) to reduce the compressive stress at the extrados of the crown due to subsequent construction; (2) to reduce the stress at the crown extrados due to rise in temperature (the time of year was one of minimum temperature); and (3) to reduce the hazard in case floods damaged the centering. Although keeping the crown hinge open would tend to reduce the effect of progressive shrinkage on stresses and to lessen the effect of movements of the abutments, it was felt that these considerations did not outweigh the benefits of closing the hinge.

MOVEMENT OF ABUTMENTS OBSERVED

Duralumin plugs were set into the concrete of the abutments, located as shown in Fig. 5, and on them transit observations were taken to determine both the rotation and the horizontal displacement of the abutments. Plugs 3 and 7 were referred to a precisely established reference line and served to show horizontal movements and vertical settlements. Plugs 1 and 2 and also Plugs 5 and 6, which were set a predetermined distance apart, afforded a plumb reference line from which transit observations of the rotation of each abutment were made and recorded. Plugs 4 and 8 were set so that deflections of the deck might be observed.

Plugs were located on both the north and south faces of the abutments. The movements recorded in Table II are the average of those noted at these faces. During decentering, the north and south faces were watched simultaneously by transitmen to observe the horizontal and rotational displacements.

ARCH RIB IN TWO-HINGED CONDITION
Longitudinal Steel at Crown Welded and Hinge Concreted

As shown by the condition at Stage 1, part of the weight of the abutments was unintentionally carried by the centering and the load was sufficient to crush the timbers at the point of contact. By the use of temporary hinges, the stresses due to these movements were largely eliminated, which would not be possible in a fixed arch.

UNIT DEFORMATION IN HINGES MEASURED

In Fig. 5 are shown the locations of the plugs set into the temporary concrete hinges, on which extensometer readings were taken to determine the stresses in the hinge concrete. All plugs were set 40 in. apart, and measurements were made between scratches on them with an inside micrometer.

Laboratory tests at the University of California on hinge specimens of 1:2:3 concrete with 3 per cent spiral reinforcement demonstrated that, up to a unit stress of 2,500 lb per sq in. of cross section, a unit deformation of 0.0008 in. per lin in. of specimen ensued, and that the behavior of the specimen closely followed Hooke's law. Accordingly, 2,500 lb per sq in. of hinge cross section was

adopted, to be applied, however, as direct central load. There was some doubt as to whether pure pin action would occur. Measurements on the hinge plugs proved that considerable bending moments not precisely designed for did take place. Measurements taken on the crown hinge of the south railway rib are given in Table III and are typical of the other measurements taken.

If the straight-line theory of stress distribution across the section of the hinge could have been assumed, it would have been possible to compute from the observed deformations what the corresponding stresses were. Knowing the value of the thrust and the hinge cross section, the stress due to direct compression was known. The latter subtracted from the stresses computed from the deformations would then give the stresses due to bending alone.

In default of a valid theory of stress distribution across the hinge section, an idea of the magnitude of the bending stresses had to be obtained indirectly by noting what

Crown Steel Lapped and Welded

stress must prevail in the test specimen to create the maximum observed deformation. As shown in Table III, this occurred at the crown hinge at the extrados or compressive side at closure and amounted to 0.00223 in. per lin in. When superimposed on the stress-strain curve of the test specimen, this deformation corresponded to a stress of 4,100 lb per sq in., at which the specimen no longer rigidly followed Hooke's law. There was a very close analogy as to details of design between the laboratory specimen and the hinges as constructed, but the specimen was subjected only to a direct central load, whereas the hinge in the arch was more severely stressed because the thrust acted eccentrically, as the observed

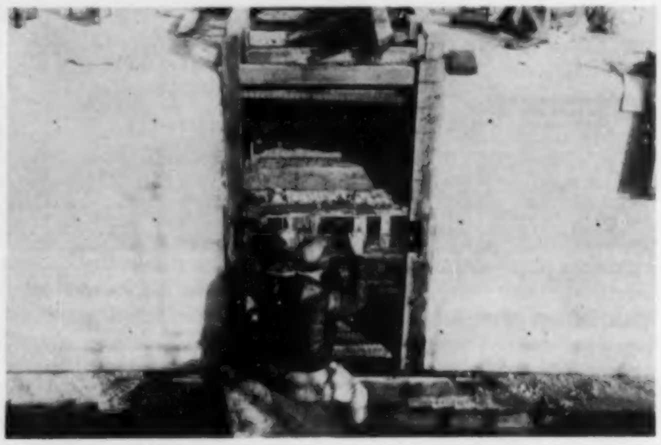
Table III. Measured Deformations of Crown Hinge in South Railway Rib

T = Tension C = Compression

Time	Deformation Between Plugs, in Inches per Inch		
	Extrados Plugs 99–100	Neutral Axis Plugs 98–97	Intrados Plugs 96–95
Immediately before decentering	T 0.0000257	T 0.0000266	C 0.000112
Immediately after decentering	C 0.0014130	C 0.0004770	T 0.000366
At hinge closure	C 0.0022300	C 0.0006900	T 0.000773

deformations show. From this comparison with the specimen, it can be deduced that the stress was considerably in excess of the assumed 2,500 lb per sq in. On the basis of a modulus of elasticity of 3,500,000 lb per sq in. for concrete and of the maximum observed unit elongation of 0.00223 in. per lin in., the straight-line distribution would yield 7,800 lb per sq in. as the maximum fiber stress. Subtracting 2,500 lb from this gives a bending stress of 5,300 lb per sq in.

Of course the total stress and the extreme fiber stress due to eccentricity of thrust, found as explained in the preceding paragraph, are speculative. It would be valuable to be able to obtain even more accurately, and in advance of construction, the amount of bending stress to be expected in such a hinge.



Taking Extensometer Readings on the Crown Hinge

From Table III it will be observed that the rib, while still resting on the centering, is indicated as being in tension both at the extrados and at the neutral axis. This phenomenon is attributable to shrinkage. The restraint of the centering prevented normal shrinkage action, which would cause the arch axis to shorten, thereby inducing compression at the crown extrados and tension at the intrados. The sagging of the rib being prevented, the rib shrinkage was translated into an inward pull of the abutments, which in turn caused a cantilever action in the rib and the customary tension on the upper fiber of a cantilever.

Observations of deformations at the abutment hinges were continued throughout the two-hinged stage. But as interest is mainly in the behavior of the three-hinged arch in accordance with design assumptions, the observa-

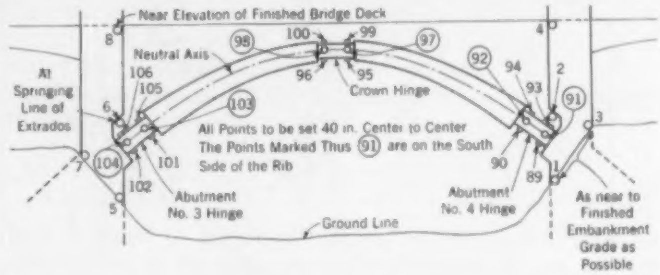


Fig. 5. Location of Plugs Set for Extensometer Readings on Hinges and to Determine Abutment Movements

tions at the two-hinged stage are not discussed here. After all hinges were concreted in, hinge plugs were set, so that changes in the rib with the lapse of time might be measured.

The Fourth Street Viaduct was designed by Merrill Butler, Assoc. M. Am. Soc. C.E., Engineer of Bridges and Structures of the City of Los Angeles, to whom credit is due for the series of field measurements taken to observe the action of the abutments and the temporary hinges.

A Formula for Border Strip Irrigation

To Give Relation Between Time of Application, Length of Strip, and Soil Permeability

By R. D. Goodrich

Member American Society of Civil Engineers
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Where conditions are favorable, irrigators apply water to crops by flooding the land in long strips between low dikes or borders extending at right angles to the irrigation ditch carrying the water. The land between the borders must be level so that the water will spread in an even sheet from end to end of the strip. This plan is economical both in labor and water, but the irrigation head must be sufficient so that the water will reach the lower end of the strip before too much has been wasted at the upper end by percolation below the root level. From some experiments in Idaho, Professor Goodrich has evolved an equation which may be helpful in determining the time-length relationship for such irrigation.

When land is to be irrigated by flooding, it is frequently divided into long, narrow strips separated by low borders from 6 to 12 in. high. In laying out fields in "border strips" for the economical application of water by this method, the engineer is usually guided by experience based on successful results which have been obtained by others. In their work on *Irrigation Engineering* the late A. P. Davis, Past-President Am. Soc. C.E., and Herbert M. Wilson state that the "strip should be of such length, and the head of water used of such volume that the water will reach the lower end of the strip before much water has had time to waste into the subsoil on the upper end where it is turned on. Thus the details must be worked out with reference to the character of the soil, the slope, and the head of water available."

In various works on irrigation, information is to be found as to the general practice followed in determining the dimensions of border strips for various irrigation heads, but the only attempt to give a definite relation between the elements of area of strip, rate of flow, and time required to cover a given area, which has come to my attention, is to be found in the recent book on *Irrigation Principles and Practices*, by Orson W. Israelsen,

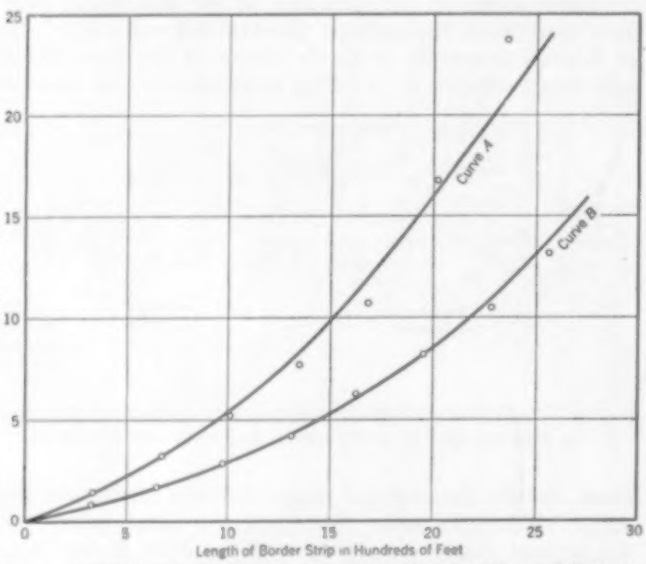


Fig. 1. Time-Length Relationship in Border Strip Method

M. Am. Soc. C.E. From a further analysis of the data given in this book, an attempt is here made to develop a simple and practical formula to express the relationship between the three elements named for the design of border strips.

The desired relation is expressed by Dr. Israelsen by a differential equation which appears to be entirely rational. However, the method of analysis that will be followed here is to find an empirical equation which will express the tabulated data with sufficient accuracy for practical purposes. The equation will then be differentiated and compared with the original differential form to show any possible correlation between the two.

Data found on page 105 of Dr. Israelsen's book are given in Table I and Fig. 1, which show a comparison of observed and computed values for the time required to irrigate different lengths of two border strips. The first experiment was with a tract of clover on gravelly soil, the strip being 49.5 ft wide and the irrigation head, 2.28 cu ft per sec. The second was with a tract of alfalfa, also on gravelly soil, the strip being 92 ft wide and the head, 7.0 cu ft per sec.

Table I. Comparison of Observed Time Required to Irrigate Two Strips of Land near Rigby, Idaho

Empirical Relationship Shown by Curves A and B in Fig. 1

TRACT A

Description: Clover tract on gravelly soil; width, 49.5 ft; flow, 2.28 cu ft per sec

Item	Division Number						
	1	2	3	4	5	6	7
Length of strip, in hundreds of feet .	3.37	6.74	10.11	13.48	16.85	20.20	23.59
Observed time, in hours	1.37	3.20	5.20	7.70	10.70	16.70	23.70
Computed time, in hours	1.42	3.24	5.55	8.43	12.05	16.45	22.00
Error	+0.05	+0.04	+0.35	+0.73	+1.35	−0.25	−1.70

TRACT B

Description: Alfalfa tract on gravelly soil; width 92 ft; flow, 7.0 cu ft per sec

Item	Division Number							
	1	2	3	4	5	6	7	8
Length of strip, in hundreds of feet . . .	3.27	6.54	9.80	13.07	16.34	19.60	22.87	25.66
Observed time, in hours. . .	0.75	1.66	2.83	4.25	6.25	8.25	10.50	13.25
Computed time, in hours. . .	0.73	1.66	2.84	4.30	6.12	8.34	11.05	13.88
Error	−0.02	0	+0.01	+0.05	−0.13	+0.09	+0.55	+0.63

CURVES OF EXPERIMENTAL RELATIONSHIPS

The empirical equation derived is:

$$qt = KBLC^{L} \quad . \quad . \quad . \quad . \quad . \quad . \quad [1]$$

in which

t = elapsed time, in hours, after turning on of water

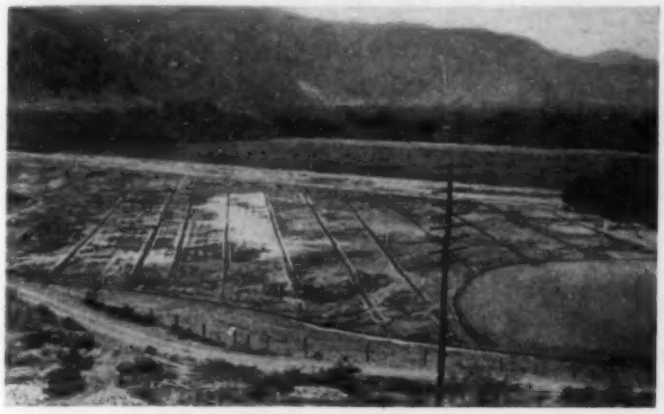
q = rate of application of water, in acre-inches per hour or cubic feet per second

B = width of border strip, in feet

L = length of strip, in hundreds of feet

C and K = constants, the values of which are to be determined by experiment for any given locality, soil, and crop

For the clover tract the value of the constant K is 0.017, whereas for the alfalfa tract it is 0.015. For both tracts the value of C is so nearly 1.04 that this average value is



IRRIGATION BY BORDER METHOD, SANTA ANA VALLEY, CALIFORNIA
Photograph by A. L. Fellows

used for both curves. As shown in Table I and in the curves of Fig. 1, the results agree quite well. In the first experiment (Curve A), the average error in the computed lengths of time is 6 per cent and the maximum is about twice that amount. In the second experiment the agreement is somewhat closer, the average error being $2^1/_4$ per cent and the maximum about $5^1/_4$ per cent.

Equation 1 will now be compared with Dr. Israelsen's general differential equation, which is:

$$qdt = ydA + pAdt \quad . \ . \ . \ . \ . \ . \quad [2]$$

in which

A = area, in acres, covered by the water in a given time, t

p = rate in inches per hour at which the water percolates into the soil

y = average depth of the water as it flows over the border strip

This equation states that the volume of water entering the border strip during the time dt is equal to the increase in the volume of water due to the flow over the additional area, dA, of width B and length dL, plus the

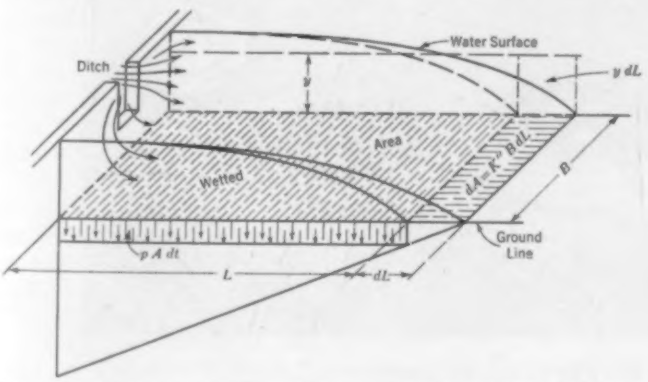


FIG. 2. GRAPHIC PRESENTATION OF BORDER STRIP METHOD

BORDER IRRIGATION OF ALFALFA, SACRAMENTO VALLEY, CALIFORNIA

volume which percolates into the wetted area, A, during the same time, at the rate of absorption p. These relations are indicated graphically in Fig. 2.

Now if in the empirical formula, Equation 1, q and B are held constant as in the two experiments described, and this equation is differentiated with respect to t and L, it will become:

$$qdt = (KBC^L)\,dL + (KBC^L \log\ C)\,dL. \quad . \ . \ . \quad [3]$$

Next, y is taken equal to $K'C^L$, and dA equal to $K''BdL$. Then the first term of the right-hand member of Equation 3 is the increment in the volume of water in the border strip due to flow over the area, as it is in Equation 2. This can be shown by considering K equal to K' multiplied by K'', substituting these values in Equation 3, and reducing this term to that in Equation 2. Here K'' is a constant that depends upon the units in which A, B, and L are measured, and K' depends on units of measure and experimental conditions, as explained later. That is,

$$(KBC^L)\,dL = ydA \quad . \ . \ . \ . \ . \ . \quad [4]$$

Since seepage through soil may be considered similar to flow in a very large number of capillary tubes, and since the velocity of flow in a capillary tube is directly proportional to the loss in head per unit length, it follows that the average rate of penetration, p, of water into the soil at any instant will vary with the average depth of water above the surface at that instant. Therefore p may be taken equal to the rate of change of y with respect to the time. Hence

$$p = \frac{dy}{dt} = (K'C^L \log\ C)\frac{dL}{dt} \quad . \ . \ . \ . \quad [5]$$

If A is equal to $K''BL$, these values of p and A may be substituted in the second term of the right-hand member of Equation 3, making it identical with the corresponding term in Equation 2. Therefore the volume of water which percolates into the ground in the time dt is given by the following equation:

$$pAdt = (KBLC^L \log\ C)\,dL. \quad . \ . \ . \ . \quad [6]$$

It appears from this analysis that Equation 1 is more than an empirical relation and that it has in fact a rational foundation.

The values of the constants C and K were obtained by plotting values of log t/L against values of the independent variable L and then drawing a straight line so as to average the points. The slope and intercept of this line

give values of $\log C$ and of $(\log K + \log B - \log q)$, since Equation 1 when written in logarithmic form is:

$$\log t/L = \log K + \log B - \log q + L \log C. \quad [7]$$

As t, L, B, and q are all known from the data of the experiment, the logarithms of C and K, and hence C and K themselves, are easily found.

As obtained from the two curves, the actual values of

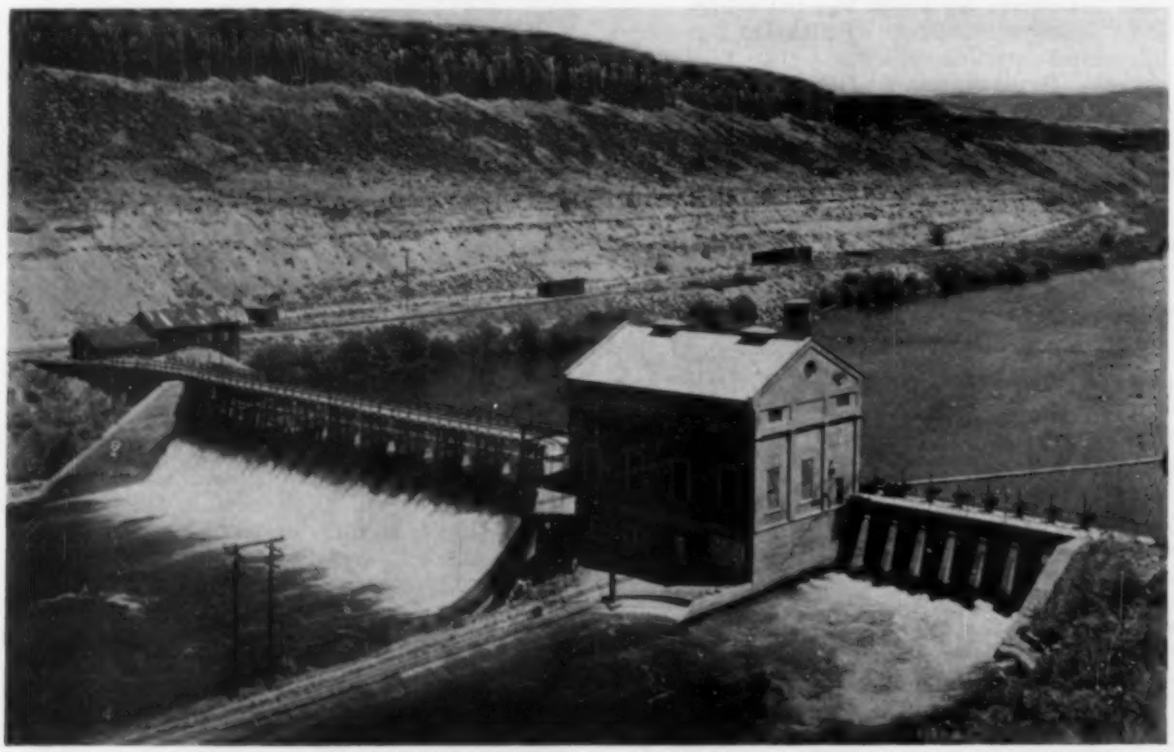
PREPARING FOR COTTON PLANTING, ARIZONA

C are 1.037 and 1.043. These differ from their average only by about 0.3 per cent, apparently because of similarity of soil and crop. This constant may also be influenced by other conditions, such as the roughness of the surface, the moisture content of the soil, and the rate of application of the water.

These equations show that with continuous flow on to the strips, the average depth of the water on the strips increased with the time. The soil in both experiments is described as "gravelly" and must therefore have been relatively porous. With a denser soil, if other factors remained constant, the rate of percolation would be lower and the average depth of water would increase more rapidly; hence the value of C would be greater. On the other hand, for still more open, sandy soil, the value of C would be smaller and might even be less than unity. In the latter case there is evidently a theoretical limit to the length of strip that can be irrigated. This length can be found by placing the first derivative of Equation 2, that is, the right-hand member of Equation 3, equal to zero. Then L will equal $-1/\log C$. Since $\log C$ will be negative when C is less than unity, L will be a positive finite distance.

Probably the values of the constant K depend upon a greater number of conditions than do those of C. Since K equals $K'K''$, it is evident that K varies with the units in which the width and length of the strip are measured, which units will determine the value of K''. For any given values of C and L, the constant K' will vary with the average depth, y, of the water on the strip. Hence it will vary with the average velocity of flow along the strip and, for a given value of q, it will also vary with the roughness of the surface and the slope.

Equation 1 shows that for any given length of border strip, the product of the time in hours required to cover the strip and the given irrigation head, measured in cubic feet per second, is a constant when the proper values of K and C are known. Therefore it should be possible to select a quantity of flow such that the soil will become saturated to the proper depth without undue loss by deep seepage. Or, for a given available irrigation head, it should be possible to select dimensions for border strips so that the water can be used with the least waste. However, to do this will require some knowledge of the values of the constants in the equation. It should be possible to determine these with little difficulty. Reports of experiments along this line, giving constants for various conditions and types of soil, would be of great value to many agricultural and irrigation engineers.



BOISE RIVER DIVERSION DAM AND POWER HOUSE, IDAHO
Boise Project of the U. S. Bureau of Reclamation

Engineers as Arbitrators of Contracts

A Knowledge of Arbitration Procedure Necessary for the Settlement of Disputes

By Daniel T. Webster

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In order to provide adequate legal machinery for the arbitration of disputes between owners and constructors, a standard clause for insertion in contracts between these parties has been prepared. The wording of this clause was determined by the Joint Conference on Standard Construction Contracts, at which the Society was represented. In cases where no agreement between the parties to a contract can be obtained by the engineer, as the interpreter of the terms of the contract, the controversy can be referred to an arbitrator for settlement. It is natural that, on engineering works, a disinterested and impartial engineer should best qualify for such duty. In this article, Mr. Webster outlines the arbitration problem and briefly explains the procedural knowledge that the arbitrator must possess to fill the post successfully.

In keeping with the traditions of his profession, as well as by training and temperament, the engineer likes to "see the job finished." The particular job on which he is engaged may be the construction of a building, a water supply system, or a public highway—or it may be the adjudication of a controversy in which he has been selected to serve as arbitrator. If the undertaking is in the field of construction, it is one for which the engineer has been trained and which he knows from experience. If he undertakes to serve as arbitrator, he will find that, in addition to his practical training and experience, he needs a knowledge of arbitration procedure, familiarity with the arbitration law under which he is acting, and the guidance of definite rules of procedure to enable him to answer the questions and solve the problems which frequently arise and which may have an important bearing on whether or not the job is to be "finished" successfully.

Qualified arbitrators and adequate rules of procedure are the two most important factors in the proper arbitration of contracts, granting the existence of a comprehensive and effective arbitration law. Like any other cure, arbitration must be properly applied or it may result in strife and bitterness instead of creating a better understanding and bringing about a final adjudication of the controversy. Arbitration means an end to litigation only when it is properly conducted, under rules which safeguard the parties and the arbitrators from procedural tangles. A casual, carelessly conducted procedure contains many elements of risk, as does an imperfectly constructed engineering project; a standardized and carefully supervised procedure practically ensures success.

THE CIVIL ENGINEER AS ARBITRATOR

The interest of the civil engineer in commercial arbitration centers more in his relation to it as an arbitrator than as a party. Engineers in their professional capacity are frequently called upon to serve as arbitrators because of their special qualifications and because of the widespread use of arbitration clauses in building and construction contracts and, to a somewhat more limited extent, in contracts for engineering projects and the construction of public works. In this article the subject of arbitration is presented from the viewpoint of the engineer as an arbitrator rather than as a party. An attempt will be made to give him information which may be of assistance with respect to the duties he may be called upon to perform in this capacity.

During the centuries that arbitration has been practiced, it has always been recognized that its success or failure depends largely on the arbitrator. If he is competent, incorruptible, and impartial, and commands the confidence of the parties to the contract, then whoever is the loser, arbitration does not suffer. When the parties depart from the custom of choosing impartial and competent men to represent them and when instead each selects a partisan or advocate and pays him for his services, leaving it to the two thus selected to choose the umpire, who alone can render a decision free from all personal prejudice, serious difficulties and dangers arise.

In constructing a national system of arbitration, the American Arbitration Association has profited by such experience and has established a national panel from which arbitrators can be drawn at will by the parties. Thus they can secure persons who have no interest whatever in the outcome of the controversy, who are not swayed by personal feeling or by their relation to the parties, and who are generally not compensated by the parties. The policy of maintaining representatives from every profession and branch of trade who can be selected to serve in controversies arising in that particular field has made it possible to secure arbitrators qualified by experience and knowledge to serve in any dispute.

CHOICE OF EXPERTS AS ARBITRATORS UPHELD

This vital principle of commercial arbitration—that of utilizing experts in different branches of industry as arbitrators within their own field—was upheld in a decision of the New York Court of Appeals in June 1930, in a controversy involving the sale of stocks. The award of the arbitrators had been set aside by a lower court on the ground that one of the arbitrators was a stockbroker and therefore disqualified to act in a case involving a stock exchange transaction. The Court of Appeals reversed this decision and unanimously sustained the award of the arbitrators. The court said in part:

> No reason has been given why this award should be set aside. It was carried out in all particulars in accordance with the agreement to arbitrate and we are now asked to hold that because a man is in a similar business, he is not qualified to act as an arbitrator, although no objection was raised to his selection as arbitrator until the day the award was made.
>
> There is no merit in the objection that one of the arbitrators was not qualified to act because he was engaged in a business similar to that of the petitioners. Knowledge of a business and the methods used therein may be of great value in reaching a just result because of the ability of an arbitrator to apply such knowledge to the facts. To hold that because of such knowledge one is disqualified to act as an arbitrator would bring about a very unjust result. There may be occasions when it is necessary to secure arbitrators with knowledge of the particular business out of which the transaction has arisen and which is the subject of the arbitration. Such

arbitrators may be much better equipped than those who know nothing about the subject matter of the arbitration;...when, as here, it is shown that a knowledge of the business has in no respect improperly influenced the decision of the arbitrators and the arbitration has been fair and just, the award should be confirmed.

This decision would seem to remove any objection that might be raised to the choice of an engineer as an arbitrator in a controversy arising out of an engineering project or involving another engineer.

STANDARD CONTRACT DOCUMENTS RECOMMENDED

When the Joint Conference on Standard Construction Contracts was called, in which the Society and seven other leading engineering and construction associations of the country participated, the result was a series of Standard Contract Documents which refer to the mutual relations of owner, contractor, and engineer in engineering construction and in building construction. The use of these contract forms is not obligatory on members of the Joint Conference; the forms are merely recommended for use by them.

The standard contract for engineering construction provides that disputes between contractor and owner shall be submitted to the engineer for his decision, and contains a further provision:

All such decisions of the engineer shall be final except in cases where time and/or financial considerations are involved, which, if no agreement in regard thereto is reached, shall be subject to arbitration.

But no standing machinery is provided by the contract, the selection of the arbitrator or of two arbitrators being left to the parties. In the latter instance the third arbitrator is chosen by the first two. The only limitation on the selection of arbitrators is that they may not be financially interested in the contract or in the business affairs of any party to the dispute. Also, they must be persons familiar, in general, with the work or the problem involved in the dispute.

In contracts for the construction of public works, the arbitration clause frequently varies with the nature of the project. According to the files of the American Arbitration Association, a notable example of such an undertaking protected by an arbitration clause is the bridge across Lake Champlain between New York and Vermont. In this contract the engineers included the following provision for the arbitration of disputes:

To minimize disputes and facilitate their prompt settlement, it is mutually agreed in Article C hereof that the Engineer shall in the first instance be the interpreter of the contract.

All decisions of the Engineer made in accordance with Article C hereof shall be final except as to the element of time and as to financial considerations involved, which, if no agreement in regard thereto is reached, shall be subject to arbitration under the published Arbitration Rules of the American Arbitration Association and pursuant to the New York Arbitration Law, the parties hereby certifying and agreeing that they have read and are familiar with said Rules and said Law.

All questions subject to arbitration under this contract shall be submitted to arbitration at the demand of either party to the contract.

By such a clause, the parties not only provided for the arbitration of any controversies which might arise, but also established specific rules and facilities immediately available, to prevent any possible delays or lapses in the proceedings.

ARBITRATION CLAUSE INCLUDED IN LARGE BRIDGE CONTRACT

The same records also disclose that the Port of New York Authority likewise protects by arbitration clauses its contracts for the construction of public works, such as that for the steel superstructure of the Outerbridge Crossing over the Arthur Kill, between Perth Amboy, N.J., and Tottenville, N.Y. In this contract the arbitration clause in part provided:

In the event of a dispute concerning the legal rights or liabilities of the parties hereunder or either of them, the parties agree to facilitate the determination of such dispute by means of a declaratory judgment of the court under the laws of the State of New York, or by submission of an agreed statement of facts to an appropriate court, or otherwise, and in good faith to reduce the cost and delay of litigation by proceeding diligently.

As to any difference between the parties where the judgment of the Engineer is not conclusive upon such question, all such differences which present questions of fact shall upon the demand of either party be submitted to arbitration in accordance with the method hereinafter provided. But no claim of arbitration or action hereunder by either party, including submission thereto, shall prejudice either of the parties with respect to any claim it or they may make in regard to the proper legal interpretation of the Contract or the determination of their legal rights or liabilities hereunder, and determination of any question of fact by arbitration hereunder shall be subject always to the determination of the legal rights or liabilities of the parties under this Contract by a Court of competent jurisdiction. If either party disputes that a difference of fact exists which is submissible to arbitration under his Contract, and such claim shall be presented to a court upon a motion to compel the arbitration to proceed, then the parties waive their statutory rights to claim a trial by jury of any of the issues involved in such proceeding. Neither party shall be bound to present such contention prior to the arbitration, but by an appropriate statement made to the arbitrators, may reserve such question for presentation and consideration upon a motion to confirm, vacate, or modify the award rendered, and upon such motion either party may ask the Court to determine any question of law relating either to the duty to arbitrate the issues presented to the arbitrators, or the legal rights and liabilities of the parties in relation to such issues. The award of the arbitrators shall, at the request of either party, be stated in such form that it presents the facts found upon the issues in dispute, and may thereupon be used for all purposes as an agreed statement of facts between the parties. It shall also state any questions of law reserved by either party at the commencement of the arbitration for the consideration of the Court.

In the clause there were further provisions to ensure the prompt and uninterrupted performance of the work in the event of a controversy, and for the selection of the arbitrators and the procedure to be followed in the arbitration.

The American Arbitration Association also reports that when the Board of Water Commissioners of Detroit prepared the contracts for the construction of its water supply system, it included a provision for the arbitration of disagreements between it and the contractor. The several contracts in which the clause was included totaled more than three million dollars.

Because of the importance of the controversies in which the engineer is usually called upon to serve as arbitrator and the large sums of money frequently involved, it is essential that the proceeding be conducted under adequate rules, designed not only to prevent deadlocks or lapses, but also to enable the arbitrators to conduct an orderly and standardized arbitration.

Whereas the engineer may be entirely familiar with the questions of fact involved in the dispute and especially qualified to make an award on the matters submitted to him for decision, he may not be so familiar with the requirements of the arbitration law under which the proceeding is being conducted. He often finds himself enmeshed in all kinds of technical questions because of the attempts of a party to introduce formal rules of evidence or to delay the proceedings. And he may be certain that the losing party will scrutinize the entire proceedings very carefully and take advantage of any irregularity or error on the part of the arbitrator that

may be made a basis for vacating the award. If, however, the arbitrator is acting under adequate procedural rules, the arbitration goes forward with speed and certainty, and a final and effective disposition of the controversy is practically assured.

Take, for example, the hearing of testimony and receiving of evidence. The arbitrators are in sole charge of these proceedings and under most arbitration laws possess wide discretion as to both the kind and the extent of the proofs which they may take. Although it is a safe rule for the arbitrators to receive all evidence offered and hear all witnesses tendered, in order that they may be sure no essential fact has been omitted, they do have the power to exclude evidence or testimony which they consider immaterial or irrelevant, or which is merely repetitious and will only result in unduly prolonging the proceedings. The same is true of the questions that may be asked of either party by the other or by the attorneys, or of attempts to introduce extraneous or improper proofs.

Concerning this point, the Rules of the American Arbitration Tribunal have been designed to give the arbitrators wide latitude. They provide as follows:

> The arbitrators shall receive all pertinent and material evidence offered by the parties or their counsel, and shall hear all witnesses presented by them. The arbitrators shall not reject any evidence which they deem necessary to an understanding of the controversy submitted to them. The arbitrators may hear arguments by counsel and receive their briefs and shall fix the time within which such briefs shall be filed.
>
> The arbitrators shall, in their discretion or upon the demand of either party, receive the testimony of parties or witnesses under oath.
>
> The arbitrators shall take adjournments on the request of a party for good cause shown and may take adjournments on their own initiative.

When an arbitrator ignores his responsibilities or acts in ignorance of the requirements of the arbitration law under which he is conducting a proceeding, there is grave danger that litigation will result from his award and that it will be vacated by the courts. In the monthly publication, *American Arbitration Service*, New York, N.Y. [discontinued in June 1933] many cases have been cited in which this has happened—where, for example, the arbitrator has made tests of merchandise without the knowledge of the parties or the attendance of all the arbitrators; where the award has not been rendered within the required time; where the arbitrator has evidenced partiality or bias and has rendered a grossly inadequate award; where the arbitrator has awarded on issues not submitted for adjudication; or where he has had some business or other relationship with one of the parties not revealed at the time of his appointment.

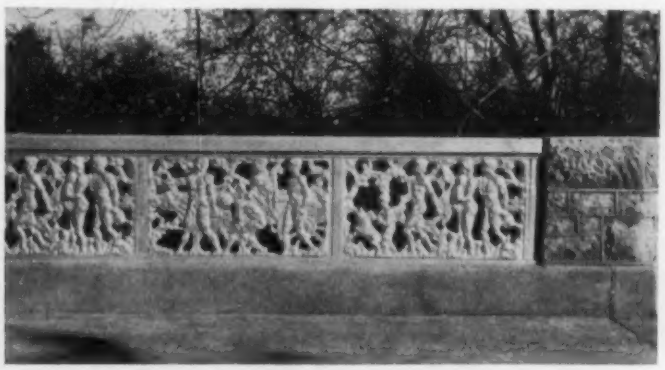
In the course of even the most informal arbitration proceedings, any number of technical questions may arise. Unless the arbitrators are familiar with the provisions of the arbitration law and are guided by adequate rules of procedure, the entire proceeding as well as the resultant award may be invalidated by a seemingly unimportant omission or error.

For example, one of the parties may submit a new or additional claim during the proceedings or after the hearing has begun, which may be outside the scope of the clause or the submission agreement. The question immediately arises as to whether the arbitrators shall refuse to admit the claim or whether it shall be decided along with the previously submitted claims. Is a new proceeding necessary for the consideration of the additional claim, or is an adjournment required? These are points on which the arbitrator must rule.

When one of the parties fails to appoint an arbitrator or a vacancy occurs in the office of arbitrator for any reason, there may be a lapse in the proceeding in the absence of a rule of procedure in this respect or unless a method is provided to fill such vacancy.

A question may arise as to the proper procedure when one of the parties seeks to reopen the proceedings after they have been closed by the arbitrators, either before or after their award has been rendered. Or the arbitrators may wish to secure an appraisal of property or additional evidence from outside sources as a guide in making their award. Again, the parties may challenge the scope of the arbitration agreement or the authority of the arbitrators, or may wish to extend the limitations of the agreement to embrace additional matters.

Such questions are constantly arising in the course of proceedings, and their correct answer varies according to the arbitration statutes of the different jurisdictions. In developing a standard practice of arbitration under the different laws of the various states, the American Arbitration Association has prepared, in addition to its standard rules of procedure, a Code of Arbitration Practice and Procedure for the guidance of parties, arbitrators, and attorneys. The code records the progress of organized arbitration in the United States, applies the rules to the different jurisdictions in which arbitrations are held, implements the existing arbitration laws, and supplies a procedure where there is no law, thus overcoming to a great extent the diversity in state arbitration laws and present differences in practice.



UNUSUAL ARCHITECTURAL TREATMENT OF 66-FT REINFORCED CONCRETE ARCH OVER CLEVELAND AVENUE, MILWAUKEE, WIS.
Steel Railing, with Cast-Iron Panels, Was Painted with Aluminum Paint. Photographs Courtesy of Charles S. Whitney, M. Am. Soc. C.E., Consulting Engineer, Milwaukee

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

Area, Center of Gravity, and Moment of Inertia of Members by Graphical Calculus

By Brent C. Jacob, M. Am. Soc. C.E.

Formerly Electrical and Mechanical Engineer, Industrial Brownhoist Corporation, Cleveland, Ohio

When irregular sections are used for members under stress, it is sometimes difficult to compute mathematically their area, center of gravity, and moment of inertia about the center of gravity. As these values are necessary in order to design members strong enough to safely carry the required load, many methods have been worked out to obtain them. A method employing graphical calculus, here described, gives results close enough for practical design, with an unusually small amount of labor.

In Fig. 1 is shown the method applied to a 60-lb American Railway Association rail, which is bounded by 14-in. radii, round corners, fillets, and sloping lines—a construction which does not lend itself to an easy mathematical solution or to some of the other graphical methods in use. The exact area, location of center of gravity, and moment of inertia of this rail are given in the Carnegie Steel Company's handbook, and a comparison of these values with those found by the method here explained is shown in Table I.

METHOD APPLIED TO STANDARD RAIL SECTION

As the rail is symmetrical, it is necessary to draw only half of it to some scale. Full size was used in the original drawing, to which the scaled dimensions of Fig. 1 refer. The section is divided into an even number of parts, such as 10, and the lengths of all the ordinates on the vertical lines from zero to 10, inclusive, are scaled and noted on the section as shown. An integral scale, n, equal to 4, was used in this case; that is, an ordinate of 1 in. is taken as 4 units. Although the length of the ordinate zero is marked 2 by scaling to the point where the curve seems to leave the vertical line, it is actually zero because the 14-in. radius has its center on the symmetrical line. Since all the ordinates are scaled as they appear and are not calculated, this value of 2 was used.

Table I. Comparison of Exact and Graphical Values for 60-Lb. American Railway Association Rail Section

Item	Area Sq In.	Center of Gravity Inches Above Base	Moment of Inertia
From Carnegie Handbook	5.86	2.13	15.4
By graphical calculus	6.00	2.13	16.5
Percentage	102	100	107

Below the half section, the 10 equal spaces of width v are divided into spaces equal to $v/3$ and $2v/3$, as shown by the auxiliary lines. This is done to utilize the parabolic theory of approximate summation.

An integral slope chart is made, with a slope scale, m, equal to 10, by taking the distance OQ on the lowest horizontal line equal to 10 units of convenient length, and erecting a perpendicular, QR, to the same scale (Fig. 1). All the division points on QR when joined with O give the desired integral slopes, such as the dashed line marked "Integral Slope 8."

AREA DIAGRAM

To make the area diagram, a horizontal line, ab, is drawn. Then, starting with the intersection of this line and the vertical line zero, the integral slope line 2.0 is drawn, corresponding to the ordinate of the rail on the zero vertical line. This slope is continued one-third of the distance to the vertical line 1. From this point a line on a slope of 4.4 is drawn to the next auxiliary line. This process is continued across the diagram to the vertical line 10, at which point the ordinate, x, measured to the same integral scale as the rail section above is found to be 4.8 units (or 1.2 in.), representing

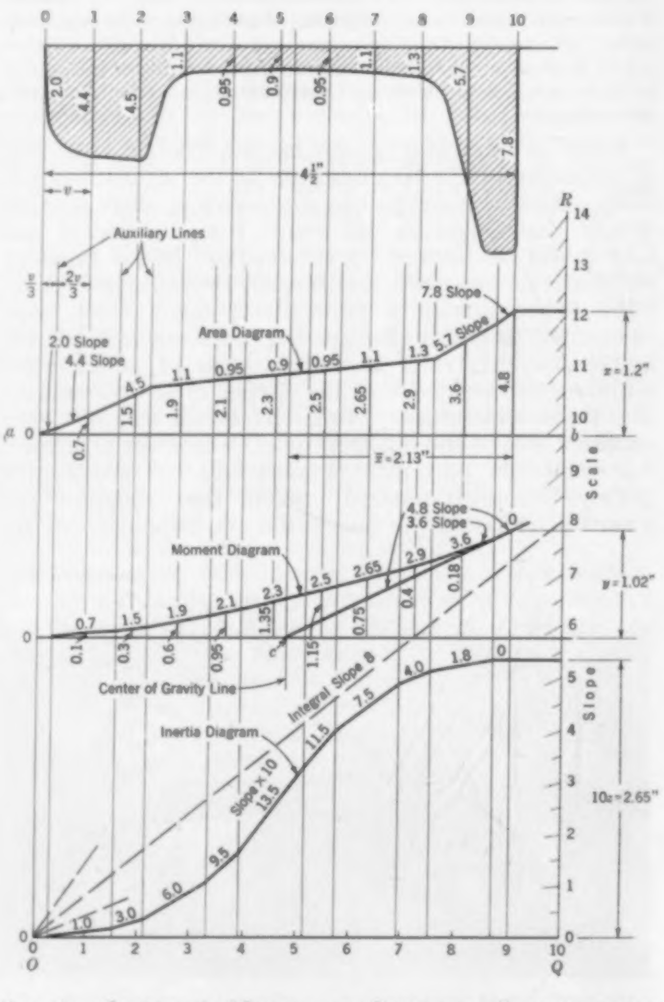


Fig. 1. Graphical Method of Obtaining Constants for Irregular Structural or Cast Shapes
Applied to a 60-Lb American Railway Association Rail Section

to some other scale the area of the half rail section. Since m equals 10, this line equals 48 square units of area, and since 1 in. equals 4 units, one square unit equals $^1/_{16}$ sq in.; then 48 square units are equal to 3 sq in. in the half section, or A equals 6 sq in. in the complete rail. This may be computed by the equation,

$$A = \frac{2xk^2 m}{n} = \frac{2 \times 1.2 \times 1^2 \times 10}{4} = 6 \text{ sq in} \ldots [1]$$

where x is measured in inches; and where k is the scale of the drawing, m the slope scale, and n the integral scale.

Then the ordinates of the area diagram are scaled with the integral scale, n, equal to 4 and are found to be zero, 0.7, 1.5, etc., as shown in Fig. 1. With these ordinates the moment diagram is constructed in the same way as the area diagram. The last slope line, 4.8, intersecting the vertical line 10, is extended back to intersect the horizontal base line at c, giving the distance $\bar{x}$ of the center of gravity line of the rail from the base. By definition this location is given by the equation,

$$\bar{x} = \frac{M}{A} \ldots\ldots\ldots\ldots\ldots\ldots [2]$$

in which the area, A, is as already determined.

The moment, M, is expressed as follows:

$$M = \frac{yk^3m^22}{n^2} \ldots\ldots\ldots\ldots [3]$$

in which the distance, y (Fig. 1), is measured in inches.

Solving,

$$M = \frac{1.02 \times 1^3 \times 10^2}{4^2} \times 2 = 12.78$$

and

$$\bar{x} = \frac{M}{A} = \frac{12.78}{6} = 2.13 \text{ in.}$$

This is a check on the location of the center of gravity.

The ordinates included between the moment diagram, the horizontal base line, and the 4.8 slope line are next scaled, with the integral scale of n equal to 4, and are found to be zero, 0.1, 0.3, etc., as shown. Since most of these ordinates are less than unity, better results will be obtained in the third, or moment of inertia, diagram by multiplying by 10 and then constructing the inertia diagram with slopes as before. The ordinate on the vertical line 10 is then $10z$, or 2.65 in., which gives z equal to 0.265 in. The moment of inertia, I, of the whole rail about its center of gravity is given by the equation,

$$I = \frac{2zk^4m^32}{n^3} \ldots\ldots\ldots\ldots\ldots\ldots [4]$$

Or, numerically,

$$I = \frac{2 \times 0.265 \times 1^4 \times 10^3}{4^3} \times 2 = 16.5$$

Needless to say, the more care taken to secure accuracy in the scale drawing, in the measurements of ordinates, and in the drawing of slopes and intersections, the better the results will be. The scales, n, m, and k, can be taken to suit conditions, and the area should be divided into an even number of equal divisions sufficient to give the accuracy desired.

MECHANICAL INTEGRATING DEVICES

Mechanical intergraphing instruments are made which eliminate the need of making the integral slope scale, OQR, and require still less work to obtain the foregoing values. There is another integraphing attachment that eliminates the integral slope scale and also the necessity for scaling any of the ordinates except the lengths x, y, and z. This is also true of the expensive machines equipped with a pointer for tracing over any curve to draw its integral.

Determining the Perimeter of an Ellipse

By HENRY F. COLEMAN
LOGANSPORT, IND.

AN exact method of determining the perimeter or circumference of an ellipse from its axes is available in calculus. However, it is a long process and involves a considerable amount of work to obtain a single value. For practical purposes, this exact solution may be used to determine the coefficient, ϕ, by which to multiply the sum of the major and minor axes, a and b, to give the perimeter of the ellipse; thus

$$\text{Rectified perimeter} = L = \phi\,(a + b). \ . \ . \ [1]$$

Values of ϕ thus calculated are shown in Fig. 1 and are plotted against the ratio of the minor to the major axis of the ellipse. As an example, if the minor axis, a, of an ellipse equals 2, and its major axis, b, equals 16, then the sum is 18 and the ratio is $2 \div 16$, or 0.125. From the curve in Fig. 1, $\phi = 1.823$, for this ratio. From Equation 1,

$$L = 18 \times 1.823 = 32.820$$

It will be noted that for a circle, where $(a + b)$ equals $2d$, the coefficient ϕ from Fig. 1 is 1.5708, or $^1/_2\ \pi$, and the familiar formula, $L = \pi d$, is evolved. Values of the coefficient ϕ may be found accurately to three places from the curve. By plotting to a larger scale, even more accurate values can be read directly.

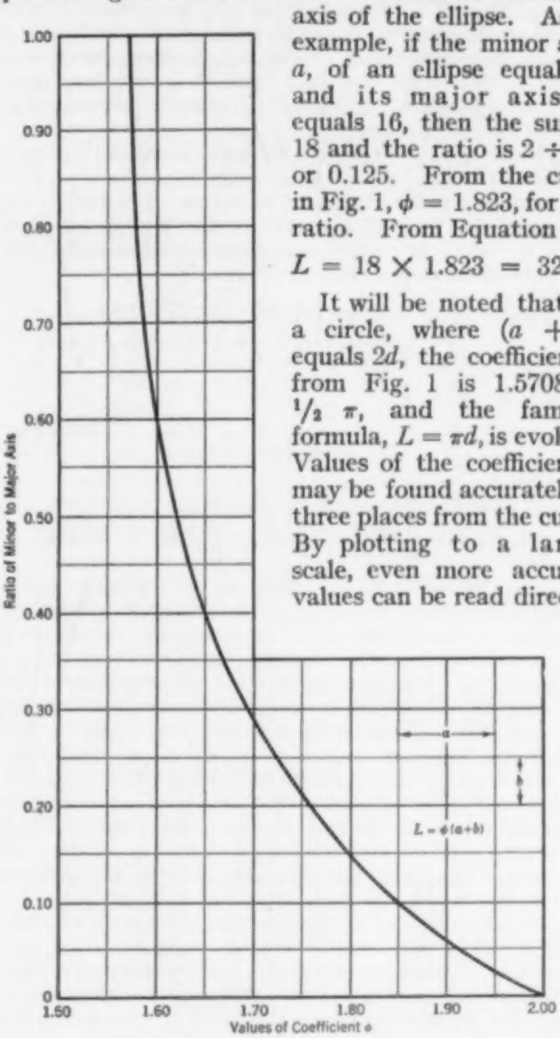


FIG. 1. COEFFICIENTS FOR PERIMETERS OF ELLIPSES

OUR READERS SAY—

In Comment on Papers, Society Affairs, and Related Professional Interests

Many Causes of Erosion

Dear Sir: In his article, in the February issue, Mr. Hoyt has stated the problem of soil erosion and watershed protection in a manner both rational and dispassionate. His article should serve to clarify the stream of public thought that regrettably has been unnecessarily muddied by certain writers who have flippantly attributed all erosion in the arid regions to the effects of overgrazing.

His statement that "overgrazing has caused the greater part of the unnatural changes in watersheds of Type B," is open to different interpretations. Unnatural erosion, even in arid regions, is seldom caused by the mere grazing of range animals. However, erosion does start in, and follow, cultural scars. It is along trails, at feeding and bedding grounds, at watering holes, and at points of repeated animal concentration that incipient erosion is in evidence. Animals are gregarious, and this fact renders control of the results of grazing difficult.

On a gentle mountain slope, otherwise adequately protected by a sparse growth of forage grasses, I have seen deep gullies started by two wheel tracks made by driving a team and wagon only once up the slope. These are sheep grazing lands, and hundreds of animals have foraged over the hills without starting unnatural erosion—it remained for the herder's wagon to do that.

In forested areas unnatural erosion seldom starts as a result of the mere removal of trees. But it does start in logways, roads, trails, and railroad grades. Since in most such areas there is sufficient rain to mature forests, the scars will heal themselves in time. However, in certain forested areas, where clay soils abound, the balance is so delicate that such cultural scars cannot heal and erosion runs rampant. Erosion of agricultural lands is invariably incipient in cultural scars.

Much can be done to prevent excessive unnatural erosion on all types of watersheds, but some rationally minded and scientifically controlled agency must be placed in charge. The new Division of Erosion of the U. S. Department of the Interior has an opportunity to render a genuine and much needed service to the Nation, but it must keep its feet on the ground.

J. C. Stevens, M. Am. Soc. C.E.
Stevens and Koon, Consulting Engineers

Portland, Ore.
February 17, 1934

Beam-Deflection Short Cuts

To the Editor: In his article in the February issue, Mr. Stewart has presented a semigraphical method of calculating beam deflections, which he offers as an improvement over the use of the area-moment analysis. This method, based on a conception of the elastic curve as a transition spiral, offers the advantage of pictorial clearness. Unfortunately, however, it resolves itself into a series of special cases for which the constants—the shape of the basic triangle and the rate of change of curvature—must either be memorized or developed for each application.

One would probably have no difficulty in this respect if he had continual use for the method, but the area-moment principle is more easily retained in the memory over long periods. This is true because the use of the conjugate beam in the area-moment method gives a perfectly general method of calculating deflections that is not complicated by the need for memorizing special cases. It can hardly be said that the various conjugate beams represent special cases, because the supports for the conjugate beam are selected by application of the area-moment propositions themselves.

Since the engineer is always interested in time-saving devices, Mr. Stewart's statement that his method is shorter than others should be favorably received. Without attempting to express a general opinion, I wish to call attention to the fact that there is no saving in time possible in the example illustrated in his Fig. 3. The use of area moments can be simplified to a greater extent than is indicated in my paper on "Wind Stress Analysis Simplified," in the January 1933 issue of the Proceedings of the Society, page 7, to which Mr. Stewart has reference.

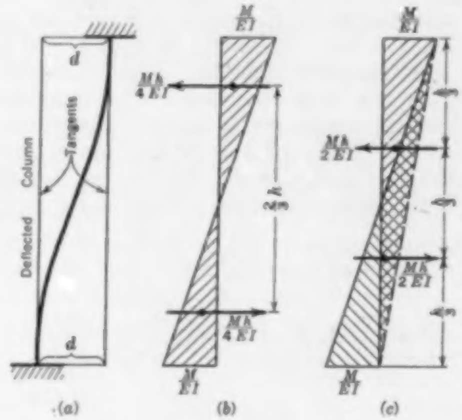


Fig. 1. Horizontal Deflection of a Fixed-End Column

In Fig. 1 (*c*), accompanying this discussion, it is shown that the deflection from a tangent at either end is the moment of the moment-area couple, or $\frac{Mh}{2EI} \times \frac{h}{3} = \frac{Mh^2}{6EI}$. Again, from Fig. 1 (*b*), the deflection is the moment of another moment-area couple, $\frac{Mh}{4EI} \times \frac{2h}{3} = \frac{Mh^2}{6EI}$. It will be noted that this second computation reproduces the exact calculations given by Mr. Stewart, although they are obtained from a different viewpoint. Commonly, there are several ways of applying the theory of area moments to a given problem, and the complication of the calculations involved will depend largely upon the choice of procedure.

L. E. Grinter, Assoc. M. Am. Soc. C.E.
Professor of Structural Engineering
Agricultural and Mechanical College of Texas

College Station, Tex.
February 26, 1934

Dimensions of Society Badge

To the Editor: In the description of the Society badge, given on page 99 of the February issue, a slight error in the statement of the dimensions occurs. If this is not corrected, it may confuse jewelers or die-makers.

The width, according to the diagram in the February issue, is $(4 + 4) - \left(1\frac{1}{32} + 1\frac{1}{32}\right) = 5\frac{15}{16}$ in., or 5.94 in., instead of 6.06 in., as mentioned in the text. The length, carried to the intersection of the cycloidal arcs, is mathematically, $5.9025 + 1^3/_8 = 7.2775$ in. However, enough is rounded off the sharp end to reduce it to the 7.25 in. referred to in the description.

D. E. Hughes, M. Am. Soc. C.E.

San Pedro, Calif.
February 20, 1934

Vacuum or Cohesion in Soils

Dear Sir: In the December 1933 issue there appears a letter by H. de B. Parsons, entitled "Vacuum in Soil Mechanics." The author's theory seems to be that the old adage, "Nature abhors a vacuum," is an explanation of the low-pressure readings in his lateral pressure tests as well as of many phenomena in cohesive soils.

A science is developed by the formulation of theories based on either experiment or experience and by checking these theories with further experiments as well as with experience. As data are accumulated, the general theories become particularized into fundamental theorems, from which future results of both experiment and experience can be predicted.

In what way can the general cause, "a vacuum," help in developing the science of soil mechanics? If it is assumed that a vacuum holds soil particles together if and when they tend to separate, it is natural to determine a correlation between the force which tends to cause movement and the value of the resistance induced. A vacuum is too indefinite an expression to use in connection with shear phenomenon, because it is a volume effect, and shear strains cause no volume changes. Merely to substitute "vacuum" for "cohesion" will help but little.

Why not continue the present method of approach? External forces induce internal resistance, and the maximum value of such resistance has been determined for many soils. It has been split into an internal friction and a cohesion, chiefly by the work of Cain. The internal friction has been considered a constant, although recent work shows large variations due to such factors as moisture, time, and direction of movement. Cohesion is a function of moisture, time or age, shape or form of grains, incipient or steady movement (potential or kinetic), colloidal content, possibly chemical content of the moisture, and other factors. Incidentally, the moisture effect has a direct bearing on capillarity and surface tension, and therefore on atmospheric pressure and gravity. Hence the cycle is complete, for Mr. Parsons' explanation of incipient or latent vacuum must also depend on atmospheric pressure and gravity.

In any new science, or in the development of a new branch of any science, there is great danger in a new terminology as well as in the incorrect transplantation of concepts from other sciences. When Coulomb published his theory of earth pressure, he did not bring in any trigonometric functions but defined the coefficient of friction as a ratio of two measured forces. In 1799 Woltmann translated Coulomb's work into German and introduced the theory that the coefficient of internal friction is equal to the tangent of the angle of natural slope. That error in assumption, possibly in translation, is not yet eliminated, in spite of the universal conclusion of soil research reports for the last fifty years, that the angle of natural slope has no relation to the value of the internal friction or internal resistance, or to the value of the lateral pressure of soils. Mr. Parsons adds his proof to Woltmann's error.

Jacob Feld, Assoc. M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
February 15, 1934

Russian Experiments on Seepage

To the Editor: In connection with the article, "Seepage Through Foundations and Embankments Studied by Glass Models," by Hibbert M. Hill, in the January issue, it may be of interest to give a brief résumé of a useful book in Russian on the subject. This work, which was published in 1931, is in two parts. The first, entitled *The Percolation of Water Through Earth Dams*, is by Prof. N. N. Pavlovsky; the second, *Experiments on Flow Through Earth Dams*, by R. N. Davidenkov, a civil engineer, supports the theory presented in the first part.

The fundamental conception of Professor Pavlovsky's work is that the hydraulic gradient—or depression curve, as he calls it—through a homogeneous earth dam is not a straight line. The upper part of the curve, extending from the intersection of the upstream face of the dam with the water surface to the vertical through the upstream edge of the crest, is concave upward and short. The central part continues downsteam to the intersection with the downstream slope in a long flat curve convex upward. The downstream part, which is steeper, extends to the intersection with the tailwater surface or to the downstream toe of the dam. Professor Pavlovsky discusses and defines formulas for the coefficient of filtration and determines the effect, on gradient and rate of flow, of various shapes of cores, upstream paving, downstream drainage systems, and various combinations of top widths and side slopes of dam. His work presents a radical departure from previous writings on the percolation of water through earth dams and on the rules for proportioning such structures.

In the second part, Mr. Davidenkov's experimental work, conducted in the hydraulic laboratory of the Scientific Institute of Land Reclamation, now absorbed by the Scientific Research Institute of Hydrotechnics, in Leningrad, is described. It shows that the shape of the theoretical curve developed by Professor Pavlovsky is in close agreement with the test curve. With an increase in the downstream slope of the dam, the depression curve is raised and the discharge is decreased. An increase in top width resulted in an increased length of path of percolation, but did not change materially the downstream shape of the depression curve. An impervious layer on the upstream slope brought the depression curve down, with a resulting decrease in discharge and an increase in the stability of the downstream slope. Impervious diaphragms placed in a dam on a somewhat pervious foundation were not as effective as a drainage system at the downstream edge of the base.

The following rules for the design of earth dams are then proposed: (1) when the foundation is impervious the use of cut-off walls is not recommended, as several shallow cross trenches give better results; (2) the top width is to be determined by practical requirements; (3) the downstream slope should gradually be made flatter toward the toe of the dam; and (4) the best drainage is in the form of a trench or embankment made of crushed stone at the toe of the dam.

Although the book has not been translated into English, a sufficient amount of information can be derived from the numerous illustrations and formulas.

I. M. Nelidov, Assoc. M. Am. Soc. C.E.
Senior Engineer of Hydraulic Structure Design, State Department of Public Works

Sacramento, Calif.
February 16, 1934

Solution by Analytic Geometry

Dear Sir: I was interested in L. S. MacDowell's article, "Passing a Curve Through a Fixed Point," in the January number of Civil Engineering.

The relation shown in his Equation 1 can be obtained by means of analytic geometry by writing the equation of the curve with the point of intersection as the origin of coordinates and the tangent line, to which the given point is referred, as the x axis.

The equation of the curve is,

$$(x - a)^2 + (y - b)^2 = R^2,$$

in which $a = R \tan \frac{\Delta}{2}$, and $b = -R$.

Then, $\left(x - R \tan \frac{\Delta}{2}\right)^2 + (y + R)^2 = R^2$

To find the value of R from this equation, substitute the coordinates of the given fixed point, D and $-L$, for x and y.

Then $\left(D - R \tan \frac{\Delta}{2}\right)^2 + (R - L)^2 = R^2$,

from which, $\tan^2 \frac{\Delta}{2} . R^2 - 2\left(D \tan \frac{\Delta}{2} + L\right)R + D^2 + L^2 = 0$

By substituting the values of $\tan \frac{\Delta}{2}$, D, and L, this quadratic equation is solved for R.

C. O. Carey, M. Am. Soc. C.E.
Associate Professor of Geodesy and Surveying, University of Michigan

Ann Arbor, Mich.
February 25, 1934

SOCIETY AFFAIRS

Official and Semi-Official

Prevailing Salaries of Civil Engineers

Board of Direction Approves Report

WHAT are the prevailing salaries of employed civil engineers? This question, often and properly asked by those in authority, has shown the need of a statement issued by some representative organization. An opportunity is thus presented for the Society to be of substantial service to a large proportion of its membership, and to others.

The accompanying report has been prepared by Arthur Richards, M. Am. Soc. C.E., of the Society's Committee on Salaries. It is a statistical analysis of the data relating to civil engineers accumulated by the committee in surveys made in 1930 and in 1933, supplemented by additional data furnished at the request of the President of the Society within the past few weeks by 31 of the Society's Local Sections. E. P. Goodrich, M. Am. Soc. C.E., Chairman of the Committee on Salaries, has endorsed the findings. With the subsequent approval of the Board of Direction these scales of salaries assume an authoritative status and as such are given publicity here.

THIS report is an effort to determine from statistical data the salaries appropriately to be paid to Civil Engineers. It is based on the surveys of salaries made by the Committee on Salaries of the American Society of Civil Engineers in 1930 and again in 1933 and reports recently received from Local Sections of the Society. It is intended to set forth the prevailing rates of salaries of Civil Engineers as of, say, March 1, 1934.

METHOD

The salaries paid 16,046 engineers and engineering assistants in the highway departments of the forty-eight states in 1930 are taken as the basis of the calculations. From numerous plottings there were determined "key" salaries. Fifty per cent of the engineers received this key salary or more. Six classifications of positions were adopted and the key salary for each was determined. Combining these, a schedule of salaries for the different classifications was set up as an index. The relation of other conditions to conditions surrounding that index schedule permits the calculation of the present prevailing rates of salaries.

REGIONS

Local conditions of ease or cost of living permit the division of the United States into regions over which these, or other, conditions indicate a variation in the salaries paid. The Bureau of Public Roads of the U.S. Department of Agriculture recognizes nine such regions and these same regions show sufficient uniformity in the salaries of Civil Engineers in the highway departments to permit them to remain unchanged in this study. These regions are:

New England: Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.

Middle Atlantic: New York, New Jersey, Pennsylvania.

East North Central: Ohio, Indiana, Illinois, Michigan, Wisconsin.

West North Central: Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, Kansas.

South Atlantic: Delaware, Maryland, Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida.

East South Central: Kentucky, Tennessee, Alabama, Mississippi.

West South Central: Arkansas, Louisiana, Oklahoma, Texas.

Mountain: Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, Nevada.

Pacific: Washington, Oregon, California.

CLASSIFICATION OF POSITIONS

Information relating to the duties, responsibilities, and salaries of the engineers of the highway departments permits a relatively simple classification of positions which further have been taken as the basis for salaries of Civil Engineers engaged in other than highway work. These classifications, comparable to the key salaries, are defined as follows:

Engineering Assistant

Duties: To assist on engineering work in field or office; make simple computations, sketches, and tracings; perform the usual work of a Rodman or Chainman.

Requirements: Education equivalent to graduation from a high school with at least two years of experience in office or field assisting on engineering work.

Junior Engineer

Duties: To perform technical work in the office or field requiring a knowledge of engineering practices and methods; make working drawings and designs and prepare estimates; make computations of a technical character; perform minor surveys and construction inspection. This classification includes the position usually called Levelman and the lower grades of Draftsman and Inspector.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus about two years of experience on engineering work; thorough knowledge of care, operation, and adjustment of engineering instruments, ability to design simple engineering structures.

Junior Assistant Engineer

Duties: To perform engineering work in office or field requiring the exercise of engineering judgment and skill; to be responsible for the surveys, plans, inspection, or work assigned to him by engineers of a higher grade; to take charge of a field party, a drafting squad, or inspection of construction. This classification includes the position usually called Transitman and the higher grades of Draftsman and Inspector.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus about three years of experience in engineering work; ability to plan operations in office or field.

Assistant Engineer

Duties: To perform important engineering work in office or field requiring the exercise of independent engineering judgment and skill; to supervise designs, drawings, and specifications; to make studies, estimates, and reports, and to supervise the work of one or more field parties or a group of inspectors on construction work. This classification includes the position usually designated as Designing Engineer.

Requirements: Education equivalent to graduation from a college or technical school of recognized standing plus at least four years of experience in engineering work of which at least one year shall have been in responsible charge; eligible for corporate membership in one of the national engineering societies or for license as a Professional Engineer; experience in surveying, design, construction or maintenance of engineering projects.

Engineer

Duties: To have charge of a minor engineering organization, or to have immediate administrative and technical charge of a

division in a large organization (comprising several squads or parties in office or field); to plan, direct, and supervise the design or construction of engineering projects; to supervise the preparation of contracts and specifications and to be entirely responsible for the engineering, construction, and business operations of his division, bureau, section, or department. This classification also includes the position usually designated as Assistant Chief Engineer.

Requirements: A first-class technical education plus about eight (8) years of professional experience in the active practice of surveying, design, construction, or maintenance of engineering projects or structures.

Chief Engineer

Duties: To have entire charge of and be responsible for entire organizations comprising more than one engineering division and engaged in more than one class of work; to determine technical and administrative policies and procedures; to be responsible for all engineering construction and business operations; and to investigate all engineering projects submitted for consideration and report regarding feasibility, cost, and economic necessity.

Requirements: A first-class technical education plus about fourteen (14) years of broad professional experience embracing a thorough knowledge of engineering office methods, field surveys, design, supervision of construction, maintenance operations, materials of construction, and general business administration.

INDEX SCHEDULE

The index schedule combines the classifications of positions with the key salary for each classification. Salaries derived from the 1930 survey are shown by the later surveys to have suffered, in common with incomes from other sources, reductions differing locally but in general variable with relation to their size; i. e., the larger reductions being observed in the higher salaries. Fifty per cent of the engineers in each position classification in the schedule received the salary shown, or more. The following is set up as the index schedule showing both 1930 salaries and the appropriate reduced salaries as of March 1, 1934:

Position Classification	1930	1934
Engineering Assistants		Not less than $1,500
Junior Engineers	$1,824	1,620
Junior Assistant Engineers	2,280	2,000
Assistant Engineers	3,000	2,600
Engineers	4,200	3,600
Chief Engineers	7,000	5,700

REGIONAL FACTORS

To the indicated key salaries there should be added or subtracted amounts indicated by the regional factors. Further, to these figures, when so adjusted, other factors are applicable, dependent upon other conditions.

The regional factors were found to be as follows:

New England	plus 3 per cent
Middle Atlantic	plus 10 per cent
East North Central	plus 6 per cent
West North Central	minus 2 per cent
South Atlantic	minus 6 per cent
East South Central	minus 9 per cent
West South Central	plus or minus zero
Mountain	minus 7 per cent
Pacific	plus 8 per cent

POPULATION FACTOR

The surveys indicate that for those employed in cities of 1,000,000 population a further increase in salary is appropriate in the amount of 10 per cent and that in New York City with a population of approximately 7,000,000 a factor of plus 40 per cent is appropriate. The data available do not indicate the appropriate additions for cities with population between 1,000,000 and 7,000,000, but probably it is somewhat proportional.

TYPES OF EMPLOYMENT

The above index schedule (adjusted by regional factor) applies to engineers engaged in work similar to highway work and resident under similar conditions. Data relative to 4,700 engineering positions of different character justify increased salaries for those employed in certain other types of work or organizations. In general those employed under civil service protection and with the benefit of pension systems, as in city employ, are found to conform to the adjusted index schedule. For those many selected individuals in other than public employment, such as those in quasi-public employment or in industry, the adjusted key salaries are observed to be increased properly up to 25 per cent and this, as in the cases of highway and municipal employment, is determined on the basis that 50 per cent of the engineers regularly employed receive this adjusted key salary or more.

EXAMPLE

A Civil Engineer engaged in highway work and defined as being in the classification designated "Engineer," is indicated by the index schedule for 1934 to have a salary of $3,600. If he works in the Middle Atlantic region that figure should be increased by 10 per cent, making it $3,960. An "Engineer" employed in the City of Philadelphia (population 2,000,000 approximately) should receive an additional 15 per cent (population factor) making $4,554 and, if in other type of employment, an additional amount equaling under some methods of employment as much as 25 per cent, totaling $5,692, might be the "key" salary.

CAUTION

Statistically the outlined system is believed to be justified, using the key salary as the primary basis. Such a system cannot take into account unusual requirements or conditions nor matters of particular circumstance in respect to those engineers upon whom the greater responsibility is placed. There is no certain method of standard appraisal of the worth of intelligence, knowledge, experience, administrative ability, enthusiasm, and activity. The above analysis, however, it is believed, affords a statistical basis to which an appraised value of these intangibles may be applied.

Arthur Richards, M. Am. Soc. C.E.
Member of Committee on Salaries

Approved March 6, 1934
Board of Direction
American Society of Civil Engineers
George T. Seabury, Secretary

Prospects Bright for Annual Convention in Vancouver

As the program and general arrangements for the Society's Convention in Vancouver, B. C., this summer take form, it becomes possible to give a preliminary sketch of this promising meeting. Several important details are already fixed: (1) while ostensibly the meeting is the Annual Convention of our Society, yet it will have the added value of being a joint meeting with the Engineering Institute of Canada; (2) as the subject for the general meeting, the important and timely question of "The Development of the Columbia River Drainage Basin" has been chosen; (3) the time has been definitely selected as July 11 to 14.

A number of Technical Divisions of the Society will hold gatherings at this time. Then too, the attendance of members of the Engineering Institute of Canada will doubtless add appreciably to the interest manifested and discussion presented at these several sessions. Society meetings in Canada have the well-justified reputation of affording unusual pleasure and profit. Many members still enjoy in retrospect the Fall Meeting at Montreal in October 1925. It is necessary to go back to 1913, however, to find an Annual Convention under Canadian hospitality; in that instance it was at Ottawa, Ontario.

Active preparations are being made by a local committee representative of the Society and the Institute in Vancouver, headed by E. A. Cleveland, M. Am. Soc. C.E. These local preparations include cooperation in the matter of the program and responsibility for the social affairs and inspection trips. Judging by the preliminary suggestions now being perfected, these will prove irresistible, not only to Society members on the Pacific Coast but to many who have been looking forward to such an opportunity of visiting the Northwest and British Columbia.

It is not too early to begin to make preliminary plans for attending this convention. Further notices will appear as details

mature. The full program will probably be determined in time for announcement in the June issue. For the Society, Vice-President Henry D. Dewell is chairman of the hard-working Western regional meeting committee, which is sparing no effort to make the Vancouver Convention outstanding in every particular.

March Society Meeting

THE REGULAR Society meeting for March, as required by the By-laws, was held on the evening of March 21 just preceding the meeting of the Metropolitan Section at the Engineering Societies Building. Director Charles E. Trout presided. After announcements by the Secretary, there being no business presented, the meeting adjourned.

Improvements Incorporated in the Year Book for 1934

AT THIS TIME of the year, members look forward to receiving the annual list of members, committees, officers, and other Society statistics. Following the innovation of last year, this will appear with the regular April issue of PROCEEDINGS as Part II, entitled "Year Book Number," to be issued April 15. By color it will be distinguished from the regular PROCEEDINGS and from preceding *Year Books*.

In general the format will resemble that of other years with one or two notable exceptions. For example, the geographical list in the back of the volume will be segregated as heretofore by states and also by grades of membership. A change will be made in the omission of the street addresses in this geographical list. For such detailed information, it will be necessary to refer to the alphabetical listing. This change involves almost a 50 per cent saving in the pages devoted to the geographical index.

Another improvement will be noted in the main or alphabetical list. Some criticism of these pages has been expressed in the past because of the fact that the type is so uniform as to be somewhat monotonous, so much so in fact as to make the identification of a particular name sometimes laborious. Dictionaries often avoid this difficulty by printing the listed word in a different type from the rest of the text, and usually in a heavier one.

A somewhat similar arrangement has been adopted for the 1934 *Year Book*. In this, the names of Members and Honorary Members are given in bold face type to distinguish them from the other names. The result is that Members' names as they are scattered throughout the pages serve to mark and stress the alphabetical position. It is expected that this innovation will greatly facilitate the rapid and convenient use of the alphabetical list.

Secondarily, this change is expected to place a certain emphasis on the senior grade of Society membership. Thus perhaps it will stimulate a large group of Associate Members, who have long since been eligible for transfer, to apply for the grade of Member. Still other changes in the *Year Book* have had the net effect of economy by condensation of considerable material in the fore part.

It is hoped that these improvements, while resulting in a net saving to the Society, will at the same time add to the utility of the *Year Book* as a practical reference work for all members.

Basic Salary Schedule Established

THE BOARD of Direction in January 1934 received word that, on work done under CWA, engineers were being paid hourly rates much lower than those being paid to skilled labor on the same work. The Board immediately initiated procedures which it hoped would be helpful. President Eddy of the Society and Executive Secretary Feiker of the American Engineering Council called upon CWA officials in Washington and proffered help, which was accepted.

It was found that a truly authoritative classification of engineers or engineering assistants with their respective prevailing salary rates would be of assistance. The duty of the Society therefore appeared clear—namely, the development of an authoritative schedule of position classifications with definition of qualifications and duties and with accompanying prevailing salaries.

Data accumulated previously by the Society's Committee on Salaries were available. In October 1933, when it was expected that the Professional Engineer's Code would provide for just such a schedule of salaries, A. J. Hammond, then President of the Society, called upon each Local Section to determine the prevailing salaries paid in its locality. Hence these data were at hand or in process of collection, and a telegram to each Local Section brought further returns by air mail.

As of March 1, 1934, therefore, it was possible to develop the report on "Prevailing Salaries of Civil Engineers," printed elsewhere in this issue. The report received the approval of the Board of Direction on March 6 and was immediately released to the CWA officials.

From Washington official notice, on March 19, advised that upon its termination on March 31, the CWA program would be replaced in urban communities with work projects. About $600,000,000 of Government funds remains available for application to the substitute measures. Of particular interest to engineers are the further comments that the pay will be at the prevailing rate for "occupation and locality."

An additional item is worthy of note, emanating from the Public Works Administration in its Release PW 7683. Under the title, "Compensation of Professional Employees on Public Works Projects," it states:

"All work to be done on Public Works projects shall be subject to the following regulation, which shall be incorporated verbatim in all construction contracts for work on the projects:

> Engineers, architects, and other professional and subprofessional employees engaged on the project shall receive the prevailing local rates for the various types of service to be rendered, except that in no case shall they receive less than the hourly rates set for the highest-paid class of skilled labor directly employed on the project.

"The State Engineer shall assist in determining satisfactory rates."

It thus appears that the Index Schedule developed by the Society, with its "key salaries" for each classification and its regional, population, and type of employment factors, becomes available at an opportune time.

Engineers' Council for Professional Development

Extracts from First Annual Report, October 1933

Brief reports of the activities and accomplishments of the Engineers' Council for Professional Development (E.C.P.D.) were given in the October and November 1933 numbers. Since then the First Annual Report of the Council has become available. Some of the important functions and ambitions are here given in the form of excerpts from the official report. Because of the deep significance of this movement in its various phases, it is hoped to include more detailed information in later issues.

DURING its first year, the Engineers' Council for Professional Development perfected its organization and made far-reaching recommendations, including (1) a program for accrediting engineering schools, (2) a minimum definition of the engineer, and (3) a suggested scheme looking to greater uniformity in the grades of membership in the professional societies.

. . . . .

Its purpose is the enhancement of the professional status of the engineer. To this end it aims to coordinate and promote efforts and aspirations directed toward the higher professional standards of education and practice, greater solidarity of the profession, and greater effectiveness in dealing with technical, social, and economic problems. Its immediate objective is the development of a system whereby the progress of the young engineer toward professional standing can be recognized by the public, by the profession, and by the man himself through the development of technical and other qualifications which will enable him to meet minimum professional standards.

The E.C.P.D. functions by studying questions within the range of its objectives, and making recommendations from time to time to the governing boards of the participating societies as to procedures that are considered of value in promoting such objectives.

It will administer only such procedures as have been approved by the boards of the participating societies and assigned to it.

. . . . .

Student Selection and Guidance

The function of the Committee on Student Selection and Guidance is to report to the E.C.P.D. schemes for the educational and vocational orientation of young men with respect to the characteristics of an engineering education and the responsibilities and opportunities of engineers, in order that only those who have the high qualities, aptitude, and capacity required to obtain intellectual satisfaction therefrom may seek entrance to such courses.

. . . . .

The following statement of objectives, approved by the E.C.P.D., was presented in the report:

1. To recommend sources of information for promising or interested high school or preparatory students, their parents, teachers, and counselors, describing the qualities and aptitudes which contribute to the successful pursuit of an engineering education and the derivation of intellectual satisfaction therefrom, the quality and quantity of the major subjects pursued in college, the technical positions normally occupied by engineering graduates, the supervisory and executive positions into which they may progress, the value of a technical education as a preparation for industrial and business pursuits, and the activities and responsibilities of the professional engineer.
2. If satisfactory occupational literature is not available, such material should be prepared by revision or compilation after consultation with the National Occupational Conference and other sources of advice.

Committee on Engineering Schools

The duties of the Committee on Engineering Schools are to report to the E.C.P.D. means for bringing about cooperation between the engineering profession and the engineering schools. As a first step in its activity, the committee has the duty of reporting to the Council criteria for colleges of engineering which will ensure to their graduates a sound educational foundation for the practice of engineering. . . . The committee recommended (1) that the Engineers Council for Professional Development undertake a program of accrediting the curricula of the various schools of engineering which are deserving of approval by the Council as representing sound and adequate instruction in various professional fields of engineering, and (2) that the basis for accrediting colleges which appears in the report be approved by the E.C.P.D. as representing the basic principles which should underlie such a program of accrediting.

In presenting these recommendations the committee pointed out that some method of accrediting engineering schools is required by force of laws governing the licensing of engineers in a majority of the states. In order that accrediting may be done uniformly, consistent with the high ideals of the engineering profession and in such a manner as to be a stimulus to the best development of engineering education rather than a deterrent to future progress through codification of certain present standards, the professional engineering societies should be prepared to administer a plan of accrediting engineering schools. The committee suggests the following principles as the basis for such a plan:

1. Absolute minimum standards of the educational process are to be avoided as likely to fetter future progress.
2. Information on important aspects of organization, administration, curricula, and standards of each school is to be assembled, the institution is to be investigated personally by a committee, and final action is to be taken by the E.C.P.D. after consideration of all aspects of the situation.

. . . . .

The Council voted to approve the recommendations of this committee and to recommend to the participating bodies that the E.C.P.D. be set up as an accrediting agency for schools of engineering. This recommendation is to be transmitted to the participating bodies when the E.C.P.D. has completed the scheme of financing such an accrediting agency.

Committee on Professional Training

The duties of the Committee on Professional Training are to report to the E.C.P.D. plans for the further personal and professional development of young engineering graduates and young men who are entering the profession without formal scholastic training. . . . It reported progress as follows:

1. A survey of junior members of engineering societies to find, among other things, some indication of plans for self-development.
2. The preparation of a personal-analysis blank to assist the individual in his program of self-development.
3. Surveys of educational facilities in areas of concentration of junior members.
4. A study of the basic objectives for future independent reading by junior members.
5. Preparation of a bulletin explaining what experience and further intellectual development are demanded by criteria to be set forth by the Committee on Professional Recognition.
6. Development of procedures for participation by joint subsidiary organizations of participating bodies in different localities.

The Council approved this report and requested the committee to proceed along the lines indicated therein.

Committee on Professional Recognition

The Committee on Professional Recognition is charged with the duty of reporting to the E.C.P.D. methods whereby engineers who have met suitable standards may receive corresponding professional recognition. . . . Upon recommendation of the committee the E.C.P.D. approved the following policy as a guide fulfilling the committee's purpose:

"The profession should establish as the goal of attainment, a series of qualifications for which the young man, whether graduate or non-graduate, may successfully strive continuously from the time he enters upon an engineering career. This goal of attainment, embodied in a certificate, equivalent to the professional degree and having a value recognizable as adequate to entitle the holder to licensing or registration in a state, should be based upon the following features:

"(a) Certification should be earned, and not granted as a mark of honor.

"(b) The code of educational qualifications should be more advanced than graduation from college, yet attainable by both college and non-college men.

"(c) The attainments should be tested individually by examination (written and oral) or the equivalent and not gaged by personal estimates and testimonials alone.

"(d) Educational qualifications should comprise scientific, technical, economic, and civic knowledge of a mature order.

"(e) The code of experience qualifications should normally make the age of certification fall between 25 and 30.

"(f) The ultimate certification into the profession should be the objective to which both the colleges and the professional societies should exert their influence. To this end the colleges should be encouraged to aid by granting the professional degree to those who have been thus certified.

"(g) The certificate into the profession should be the means by which the state registration boards would, with confidence, recognize those essentials which they observe as requisite for the registration of engineers.

"(h) And, similarly, the certificate into the profession should be prima facie evidence of technical proficiency for admission into the corporate membership of the societies.

"By such a progressive educational program involving selection of proper material, its supervised education, intimate contact with the profession during the apprenticeship stage, and the attainment of definite specified educational requirements with concurrent recognition by professional societies, educational institutions, and state laws, it is believed that an identity would be attained by which those who have not developed experience and maturity of engineering judgment would be recognized as assistants in the engineering field, and those who have attained engineering maturity would have an identity universally recognized by the profession itself and the public at large. It is believed that this definition of the engineering profession will be of immeasurable benefit."

The committee presented the results of an extensive study of the various requirements for recognition as an engineer included in: (1) the grades of membership of the various engineering societies, (2) the model law for registering and licensing, and (3) the granting of degrees by engineering schools with the view that the first step that the engineering profession must take to secure proper recognition is agreement on the minimum requirements. The committee recommended the following minimum definition which was approved by the E.C.P.D., which voted to transmit it to the participating bodies for their discussion and approval:

"The minimum qualifications for an engineer are:

"(*a*) Graduation from an approved course in engineering of four years or more in an approved school or college; a specific record of an additional four years or more of active practice in engineering work of a character satisfactory to the examining body (the examining body, in its discretion, may give credit for graduate study in counting years of active practice); and the successful passing of a written and oral examination covering technical, economic, and cultural subjects, and designed to establish the applicant's ability to be placed in responsible charge of engineering work and to render him a valuable member of society; or alternatively

"(*b*) Eight years or more of active practice in engineering work of a character satisfactory to the examining body, and the passing of written and oral examinations designed to show knowledge and skill approximating that attained through graduation from an approved engineering course, and also examinations written and oral covering technical, economic, and cultural subjects designed to establish the applicant's ability to be placed in responsible charge of engineering work and to render him a valuable member of society."

. . . . . .

The Committee on Professional Recognition is engaged in a further study of a scheme of testing the requirements for the full-fledged engineer, appreciating fully that the method of measuring achievement of the qualifications in the minimum definition of the engineer is of greater importance than the qualifications themselves.

Functions of United Engineering Trustees, Inc.

ORIGINALLY known as the United Engineering Society, United Engineering Trustees, Inc., was established in 1904 by joint action of three national engineering societies. These were joined in 1916 by the American Society of Civil Engineers to form what is now designated the "Four Founder Societies." It was formed for the purpose of holding legal title to certain of the real property of the Founder Societies and to trust funds given to these societies jointly; to advance the engineering arts and sciences in all their branches; and to maintain a free public engineering library. Thus it was organized to perform for the Founder Societies certain specific acts which are governed by contracts. To it the societies have appointed as their representatives eminent members experienced in society affairs, three from each society, twelve in all, who meet monthly except in July and August.

The annual meeting of United Engineering Trustees, Inc., was held on January 26, 1934, when officers for the year were elected: President, H. V. Coes, The American Society of Mechanical Engineers; Vice-Presidents, George L. Knight, the American Institute of Electrical Engineers and The American Society of Mechanical Engineers; and H. P. Charlesworth, American Institute of Electrical Engineers; Secretary, Alfred D. Flinn, American Society of Civil Engineers and American Institute of Mining and Metallurgical Engineers; Treasurer, C. P. Hunt, Vice-President, Chemical Bank and Trust Company; Assistant Treasurer, Arthur S. Tuttle, American Society of Civil Engineers. Members of the 1934 board are Arthur S. Tuttle, M. Am. Soc. C.E., R. M. Roosevelt, W. L. Batt, and George L. Knight (terms expiring 1935); C. W. Hudson, M. Am. Soc. C.E., William H. Bassett, Harold V. Coes, H. P. Charlesworth (terms expiring 1936); and John P. Hogan and H. G. Moulton, Members Am. Soc. C.E., and D. Robert Yarnall and Harry R. Woodrow (terms expiring 1937).

United Engineering Trustees, Inc., now has two departments established pursuant to joint actions of the Founder Societies: The Engineering Foundation and the Engineering Societies Library. It also has an administrative staff. It is titular owner of the Engineering Societies Building, with a book value of $1,987,794, a depreciation and renewal fund for the building, and trust funds for the Library and The Engineering Foundation, including the Engineering Foundation fund, the Library Endowment fund, the Henry R. Towne fund, and the Edward Dean Adams fund, totaling $1,438,295. The corporation is custodian of the John Fritz Medal fund. The Library is valued at $480,800 and increases in value at the rate of approximately $20,000 a year. The corporation and its departments are tax exempt. It has competent financial custodians and advisers and legal counsel. Its accounts are audited by certified public accountants.

Space in the Engineering Societies Building not occupied by the Founder Societies is allotted to associate societies, and the use of the meeting halls is allowed to other patrons, the revenue reducing the burden of maintenance, operation, and fixed charges. Complete information is given in the *History, Charter and By-Laws*, which may be had on request to Alfred D. Flinn, Secretary of the United Engineering Trustees, Inc., 29 West 39th Street, New York, N.Y.

Scope of Engineering Code Modified

DURING the last three months the concept of the scope of the proposed Engineering Code and the principles of its application have undergone profound change. It has been impossible therefore to keep the profession advised of developments. Not until about March 20 had principles and concepts been established sufficiently to indicate a definite policy.

The draft of the Code which went through the NRA public hearing and subsequent NRA Board conferences was developed around the concept of complete control for all engineering functions within the construction industry, regardless of how or by whom performed. It included many principles affecting employees as well as employer. This draft was rejected because the NRA ruled that:

1. In order to include all types of organizations practicing engineering as defined, it was necessary to obtain cooperative approval and to show representation for employer units providing engineering services when combined with construction or specialized installation.
2. The Code could only be an employers' code and therefore the provisions dealing with or affecting employees were ruled out.
3. The Code could not affect Federal, state, county or other political subdivisional employers or employees.

As a result of these rulings the profession was given the sharp alternative of presenting a code which would be applicable only to those engineers engaged in private practice, or of meeting the requirements as to acceptability and representation for other methods of practice and including them in its code. The latter alternative was accepted.

A decision was upheld by the NRA that the Code should include all principal competitive methods of engineering practice, whether of a thoroughly disinterested professional nature or combined with non-professional functions.

There being no national organization representative of employers providing engineering services other than the Society, which had been accepted as representative of the engineer in private practice only, a series of conferences was held with representatives of the principal engineer-construction organizations. The deputy administrator handling the Engineering Code sat in on most of these conferences. A new draft of the Code has been developed as a result and has been submitted to the NRA for new hearings.

This draft includes all engineering functions in the construction industry which are of a competitive nature, either as between engineers in private practice, engineer-constructors, or commonly competitive to both. It however excludes those engineering functions in the construction industry that are incidental to manufacturing, or in which the engineering is only incidental to highly specialized or patented process installation and is not primarily competitive with engineering as practiced under the definition of the Code.

Having been submitted to the NRA, the present draft will nevertheless require more negotiations and NRA Board conferences, and in all probability will receive other NRA rulings which will require certain modifications and changes. However, with a policy established and a basis of procedure decided upon, there is now the desirability of a continuation of open discussion and frank dissemination of information to engineering organizations throughout the country.

CARLTON S. PROCTOR, M. AM. SOC. C.E.
Chairman, Code Committee

[By action of the Executive Committee of the Society at its Meeting held on March 23, 1934, a Committee on Code was established consisting of Dean G. Edwards, New York; Frank A. Marston, Boston; Frank A. Randall, Chicago; Henry J. Sherman, Director, Camden, N.J.; and Carlton S. Proctor, New York, Chairman—all Members Am. Soc. C.E. An appropriation was made for the use of the committee.]

A Preview of Proceedings

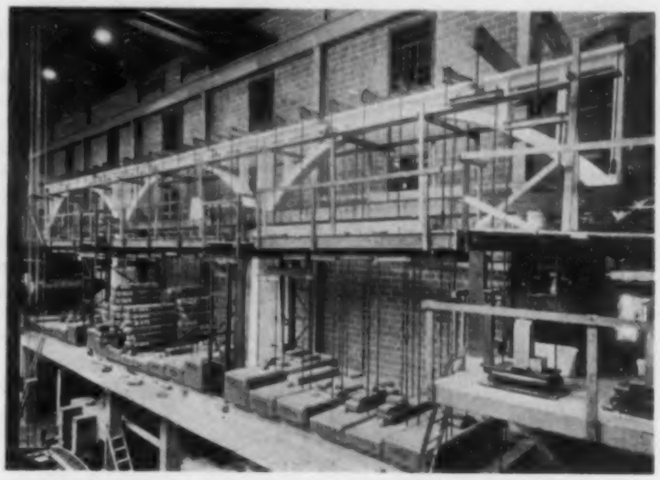
In the April issue of Proceedings *two main papers will appear: that by Hans Kramer, "Sand Mixtures and Sand Movements in River Models," announced for March but crowded out by the large number of discussions received on current papers; and that by Professor Wilson, on laboratory tests of multiple-span reinforced-concrete arch bridges, containing a description of an extensive experimental study recently completed at the University of Illinois and conclusions drawn from it.*

Laboratory Test of Multiple-Span Reinforced Concrete Arch Bridges

A most extensive experimental study of multiple-span reinforced concrete arch bridges was recently completed at the University of Illinois by Wilbur M. Wilson, M. Am. Soc. C.E., Research Professor of Structural Engineering. The structures tested were all three-span bridges consisting of arch spans 27 ft long and piers of three different heights—10, 15, and 20 ft. The arch spans were of the following types: a rib without deck; a rib with spandrel columns and high deck; and a rib with spandrel columns and low deck integral with the rib at the crown, the deck structures being tested both with and without intermediate expansion joints.

Apparatus was designed specially for the investigation and was capable of regulating and measuring movements of the supports and of weighing the reactions. Measurements of movements and forces were made with remarkable precision considering the size of the structure and the magnitude of the forces involved.

A great deal of experimental data was obtained, but it was feasible to present only a small part of the data in the paper. As a result of his analysis of the data, Professor Wilson concludes among other things that the elastic theory for the rib without deck, based on the usual assumptions, gives values for moments, thrusts, and shears in the continuous structure that agree with the measured values within the tolerance of the tests. The concrete in the arch developed approximately the same maximum unit stress as the same concrete in 6- by 12-in. control cylinders. The unit compression due to the design loading was greater for the three spans in series on 20-ft piers than for a similar single span with fixed ends for the structures where the critical stress occurred near the center, that is, for the rib without deck, and the decks with intermediate expansion joints. For the structures without intermediate expansion joints, the deck assists in resisting the moment in the central part of the structure where the flexure of the piers increases the moments. For these structures the critical section in the rib is at the springing line, where the flexure of the piers reduces the positive moment, and thereby the design load stress, when the dead-load moments at the springing line are positive. A view is shown of the low-deck structure under its maximum load.



Multiple-Span Arch Model with Low Deck
Materials Testing Laboratory, University of Illinois

The investigation described was sponsored by the Engineering Foundation and the Society, and was made by the University of Illinois in cooperation with the U. S. Bureau of Public Roads.

News of Local Sections

Iowa Section

A meeting of the Iowa Section was held in Iowa City on February 8. The two speakers of the occasion were A. H. Holt, of the University of Iowa, and Dean Anson Marston, of Iowa State College. Dean Marston discussed the subject, "Progress in the Solution of the Mississippi Flood Control Problem." On November 16, 1933, the members were addressed by R. A. Moyer, of Iowa State College, on the subject, "Skidding Characteristics of Various Road Surfaces," and by J. S. Dodds, of the same college, who discussed the relief work of the U. S. Coast and Geodetic Survey in Iowa.

Kansas City Section

A joint meeting of the Kansas City Section and of the local branch of the American Military Engineers was held at the University Club on February 17. The principal speakers were Lieut.-Col. Jarvis J. Bain and Capt. Theodore Wyman, Jr. The former outlined the world military situation, and the latter spoke on the development of the Missouri River for navigation and on the Fort Peck Dam project. After this program was presented, the Section held a business meeting, at which affairs of local engineering interest were discussed.

Los Angeles Section

On February 14, a meeting of the Los Angeles Section was held at the University Club. When dinner was over, several business matters were discussed. Then Ralph J. Reed, director, gave an account of the Annual Meeting of the Society, held in New York, N.Y., in January. After a brief recess, two interesting speakers were heard. These were Joseph Dixon, author and lecturer, who described his experiences on a trip into the Antarctic, and C. H. Purcell, chief engineer of the San Francisco-Oakland Bay Bridge, who discussed the general and engineering phases of that project. There were 160 present at the meeting.

Metropolitan Section

Various features of the Boulder Dam project were treated most interestingly at a special meeting of the Metropolitan Section held in New York, N.Y., on March 13. The general background of this great structure was given by Elwood Mead. He was followed by J. C. Hodge, who explained the fabrication of the large steel penstocks. Finally, S. C. Hollister discussed the design and analysis of stresses in the welded rings and their connections. This meeting was a joint one with local sections of the American Society of Mechanical Engineers and the American Welding Society. The attendance was about 900. At the regular meeting of the Section, on March 21, Henry Norris Russell, Professor of Astronomy at Princeton University, gave a scholarly lecture on the universe of time and space, and the possibilities of life within it. After discussion from the floor, refreshments were served and the meeting adjourned. Approximately 300 were present at this session.

The Junior Branch of the Metropolitan Section held its regular meeting on February 27. An informal address was given by Arthur V. Sheridan on the subject, "The Professional Engineers' License Law." It was announced at this session that the annual speaking contest of the Junior Branch will be held during the May meeting of the Metropolitan Section. There were 40 in attendance.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for May

One of the largest engineering projects now under construction in the United States is the Colorado River Aqueduct, which will supply water to the Metropolitan Water District of Southern California. At the present time efforts are concentrated on driving the 29 tunnels that aggregate more than 91 miles of the 240-mile route. Numerous methods of drilling, shooting, mucking, and timbering are being employed on this work, some of them for the first time. The chief engineer of the project, F. E. Weymouth, M. Am. Soc. C.E., has prepared for the May issue an informative article on the driving of the tunnels.

Another interesting article scheduled for this issue is by Charles W. Gennet, Jr., M. Am. Soc. C.E. This concerns the development of American railroad rails, stressing the larger size of rail required with increasingly heavy wheel concentrations and greater train speeds. A brief description of the manufacture of rails in this country is followed by a statement of the baffling problem of rail failures, which is being studied by a joint committee of manufacturers and users of rails. Encouraging results are promised.

For centuries the orifice has been used as a device for measuring water and has been the the subject of much experimentation and discussion on the part of engineers. In an article that has been prepared for the May issue, Wallace M. Lansford, Assoc. M. Am. Soc. C.E., presents a great deal of material on the subject of recent investigations in this field. From a study of this material, some of which has never been published, he arrives at certain definite values of the coefficient of discharge for various ratios of the diameter of orifice and pipe.

Canadian canoeists use a mixture of pitch and tallow to make their birch bark canoes water-tight. Engineers on the Western Section Irrigation System of the Canadian Pacific Railway Company have adopted a similar mixture to seal the leaks in the Winona Flume and to prevent the erosion that wears off the galvanizing. The success of the treatment is recorded in an article for the May issue by Robert S. Stockton, M. Am. Soc. C.E.

About twelve years ago an earth and rock-fill dam was built on the Hetch Hetchy Aqueduct to form a forebay for the Moccasin Power House. The unusual feature of this dam is its articulated concrete core wall, which is grouted into the rock foundation. The dam was built by, and has since been under the observation of, M. M. O'Shaughnessy, M. Am. Soc. C.E., who has prepared an article for Civil Engineering on the construction methods used, the cost of the dam, and problems of maintenance.

Evolution of Draftsmen's Tools

At Cornell University in June 1934 will be held the annual meeting of the Society for the Promotion of Engineering Education. At that time there will be exhibited a historical display of all kinds of instruments used in connection with drafting. It is to be an exhibit showing the evolution and background of the development of draftsmen's tools.

The committee in charge of this project seeks the cooperation of those who are willing to loan old instruments, tools, or devices for the period of the exhibit or have knowledge of possible sources whence such instruments may be secured. Where the instruments themselves are not available, photographs and cuts will be gratefully received. Dean Kimball of Cornell University has offered proper and safe display facilities, and the articles loaned will be returned promptly to the owners at the close of the exhibit.

Lists of articles available, suggestions, or helpful information which will make this exhibit worth while should be sent to the chairman of the committee in charge, F. W. Ming, The Polytechnic Institute of Brooklyn, Brooklyn, New York.

Engineers as Viewed by a Writer

An editorial column in the Scripps-Howard newspapers is written by Dr. Harry Elmer Barnes, prominent educator and writer. In this column he has made observations on many subjects, including the engineer, who comes in for the following, which appeared in the *New York World-Telegram* for November 29, 1933;

"Those things which separate our civilization from the everyday life of George Washington's times we owe primarily to the scientists and engineers. Our material civilization is almost exclusively their product. Without them we would not have our modern machinery; our transportation devices, from the railroad to the airplane; our telephone, telegraph, cable or radio; our bridges, subways, and tunnels; our factory system; urban life as we know it; our modern sanitary engineering, which alone makes city life possible; and many other factors which are characteristic of the world we inhabit in the twentieth century.

"Yet these scientists and engineers have received only a slight material reward for their efforts. The money-makers have exploited the intelligence and industry of the scientists and engineers and have thus built up the great fortunes which characterize our age and its concentration of wealth. Engineers may receive only a few thousand dollars for all the labor involved in designing a great plant and its machinery or in building a great railroad system, while those who supply the capital may make hundreds of millions of dollars out of such an enterprise.

. . . .

"When the depression came along and building and construction all but ceased, the engineers of the country were ruthlessly abandoned by those who could not have made a cent had it not been for the engineering brains they had exploited. It has been estimated that in New York City alone there are no fewer than 20,000 unemployed engineers.

"Much responsibility rests upon the engineers themselves. No other group is so absolutely indispensable in modern life. This is only another way of saying that no other group is so easily able to take care of itself if it would develop the proper organization and esprit de corps.

"If the engineers go on leaving themselves absolutely at the mercy of capitalism they are likely to be treated in the future as they have been in the past."

John Ripley Freeman, John Fritz Medalist

Many requests have been received for the printing of the address delivered by Thaddeus Merriman, M. Am. Soc. C.E., on the occasion of the posthumous award of the John Fritz Medal to John Ripley Freeman, Past-President and Hon. M. Am. Soc. C.E., on January 17, 1934, during the Annual Meeting of the Society. Not only for the benefit of those who made the requests but also for others unable to be present at the ceremony, the address is here reproduced in full.

"The achievements of an individual define his quality and his worth, while the recognition of accomplishment by his contemporaries is the highest reward that can come to him. So it is today a rare privilege for us to inscribe on the scroll of the John Fritz Medal the name of John Ripley Freeman, Past-President and Honorary Member of the American Society of Civil Engineers.

"His accomplishments in the fields of civil engineering and of fire insurance are indelibly written in the minds of all who knew him and are recorded in detail in the memoirs of the Society. Largely did he strive and largely did he achieve. His name is known in all quarters of the world. He has made for posterity to ponder a record of singleness of purpose, of truest endeavor, and of highest attainment.

"As we pay our tribute to, and voice our esteem of the medalist of 1934, we might rest content with a categorical recital of the things he has done. That of itself would do him honor, but it would not be enough. Than he, few indeed

have had a record which in importance and diversity is longer or more impressive. For the good of our own souls, therefore, and as a proper discharge of the honor we confer, it is incumbent upon us to go further and to inquire into the characteristics and the qualities which were at the foundation of his success. The future will be unable either to do this or to evaluate the influence which Freeman and his work exerted on engineering thought and practice. In these respects it is our duty to make the record as complete as best we may.

"By nature Freeman was methodical and energetic. A tireless student and worker, he always had time to learn and to do. No task was too difficult or too long and to each he brought a judgment in which a knowledge of practice and experience was blended with an understanding of the analyses and methods of mathematics and of abstract theory. He was a master of the logic of engineering, and thus, above the level of the mere technician, he could distinguish between fact and hypothesis, between truth and surmise. He was a keen analyst. His fertile imagination, augmented by a remarkable memory, enabled him to visualize, to plan, and to execute.

"On the preceding counts alone he was a man of rare qualifications; yet his attainments went even further. In the broad domains of physics and chemistry, which touch at every hand on engineering, he kept abreast with developments as they came. He traveled extensively and learned to appreciate and to understand the habits of thought of other peoples. His friends in all walks of life were legion. To them he gave of himself; from them he learned and constantly widened his horizon. In the communities where he lived he played his part in local enterprises, activities, and promotions. His light was not hidden. He held public office. He knew his representatives in Congress and so was enabled to make his influence felt on matters relating to the public welfare. And, weaving as a thread through these many activities, was his great interest in engineering education, whether at the universities or through the engineering societies. For forty years he was a member of the Board of Trustees of the Massachusetts Institute of Technology.

"In the field of fire insurance he made his mark as prominently as in engineering. Under his direction and guidance as president of the Associated Factory Mutual Fire Insurance Companies of Providence, R. I., the science of protection against fire was brought to a high state of perfection in all of its details and made these companies preeminent in the field of factory insurance. Combining the qualities of the engineer and the insurance executive, he constantly sought to safeguard human life and to prevent losses by fire. His writings on these subjects were fundamental and remain as classics of the literature.

"In order to interpret this brief recital of the accomplishments and the characteristics which stamped Freeman as a man among men and as an engineer of unusual attainment, it is important that we seek for some clue to the inspiration that fixed for him his goal and set on high the star that he so consistently followed and which led him ever onward.

"Born in 1855 of a long New England ancestry, he first saw the day on a farm in Maine. Soon after graduation in 1876 he came for ten years under the tutelage of Hiram F. Mills, that master of hydraulics. Here he was thrown into close contact with the best engineering minds of the day in the persons of Boyden, Davis, Francis, and Storrow. In some one of these or in them all collectively he undoubtedly found the ideal he thereafter emulated and the inspiration by which he trained his guidon. No one could have gone to a better school. The courses of apprenticeship it offered were those in accuracy, in perseverance, in hard work, and in careful thought. Freeman passed them all *summa cum laude*, and they became his habit and his mode of life.

"The influence of John Ripley Freeman on engineering thought and practice was twofold. In the first place his friendly, constant, and untiring interest in the welfare and the advancement of the young engineer won the confidence and esteem of those whose lot it was to know him. Many of these came to recognize the inward nature and true value of the principles of which Freeman was a living exponent, and they will carry them on with ever growing influence and constantly increasing effectiveness. In the second place, his contributions to the literature of engineering were of a fundamental nature and will endure. Many of these were models of keen analysis based on a thorough compilation of all available data. His reports on the water supplies of New York and San Francisco are to be emulated for width of scope and breadth of vision. His papers contributed to the engineering societies on the hydraulics of fire streams, on flood control, and on other subjects all had a profound effect in clarifying, crystallizing, and advancing the technical practice of engineering. His report on the levels of the Great Lakes further extended the principles and the style of his work, while finally, his masterful book on earthquake effects at one stroke brought to the fore a subject that will never be outmoded.

"Freeman was of an inquiring turn of mind. From Mills and those others of his earlier years he learned the art of experimentation. Long before these modern days in which research has become a word to conjure with, he did research of high value and order. His vision of the needs of the future in the field of hydraulics led him to assemble, to translate, and to publish that valuable collection of papers under the title of *Hydraulic Laboratory Practice*. In season and out he advocated and urged the need for a National Hydraulic Laboratory and these efforts finally came to fruition when the necessary funds were appropriated by Congress. And it was because of this installation that other laboratories for hydraulic research were almost immediately established, notably that of the Mississippi River Commission at Vicksburg, Miss.

"Early Freeman recognized that a mere laboratory without trained investigators could not lead to successful results. He therefore, in 1924, contributed three funds, the incomes of which were to be used for the purpose of enabling young men of promise to study in European laboratories with the view of qualifying them for experimental work here at home. One of these funds is administered by the Boston Society of Civil Engineers, one by the American Society of Mechanical Engineers, and one by the American Society of Civil Engineers. Under these three funds 19 young men have gone abroad and returned. They are now engaged either in teaching or in experimental work in hydraulic laboratories.

So has the seed grown, and its end is not yet. Thus did Freeman in a definite and practical way further the science of hydraulics and make a profound mark on the trend of American research. Thus did he discharge his obligation to the profession in which he gained honor and distinction.

"The award of the John Fritz Medal is an enduring token of the worth of John Ripley Freeman. It is a formal acknowledgment of his accomplishments and makes of them a record for the generations yet to come. Divine Providence willed that he should not live to receive this honor in person and so, today, we who knew him cannot help but feel that the award so richly earned partakes also of the nature of a tribute to his memory."

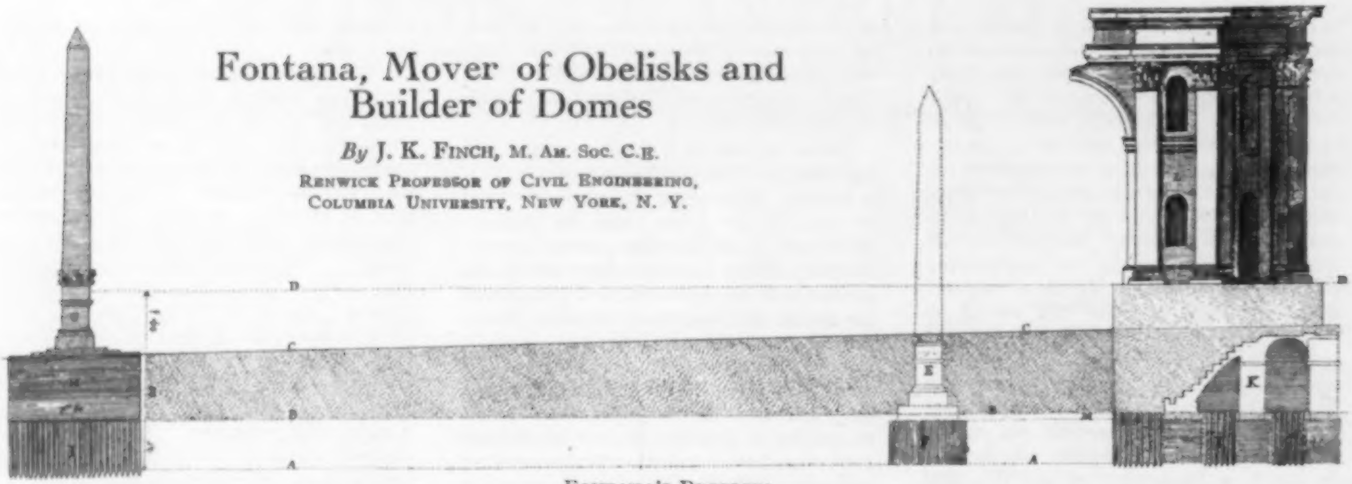
53-Year Index to American Water Works Association Publications

THE AMERICAN Water Works Association announces that there has been prepared, under one cover, a complete index to include 53 years of the *Proceedings*, the *Journal*, and other publications of the association from the earliest issue, that of 1881, through 1933. Such a complete and comprehensive index was badly needed and will facilitate the finding of articles published by the association during its existence. The index is classified according to subject matter and authorship. It is also cross indexed and thereby makes all the voluminous but scattered information contained in the association publications readily available for reference.

This index has been compiled under the direction of A. V. Ruggles, M. Am. Soc. C.E., Assistant Secretary of the association. It contains 200 pages, bound between stiff buckram covers, and has been printed in a limited edition for sale at $2.50 per copy. Orders should be addressed to the American Water Works Association at 29 West 39th Street, New York, N.Y., and will be filled on or about April 1.

Fontana, Mover of Obelisks and Builder of Domes

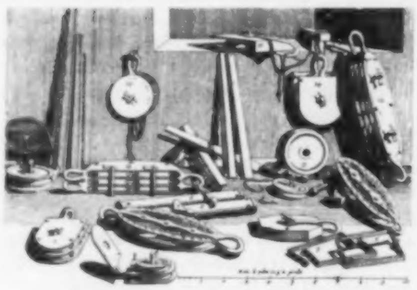
By J. K. FINCH, M. AM. SOC. C.E.
RENWICK PROFESSOR OF CIVIL ENGINEERING, COLUMBIA UNIVERSITY, NEW YORK, N. Y.



FONTANA'S PROBLEM

IN a fascinating book, the *Problem of the Obelisks*, published in 1923, the veteran Egyptian archeologist and expert on ancient construction in the Nile Valley, Dr. R. Engelbach, tells of the work of the obelisk expert in the day when these monoliths were in great demand by the kings of ancient Egypt. Thanks to Dr. Engelbach's researches, the methods of quarrying are now fairly clearly known. The transportation down the Nile in barges from the quarries at Aswan was pictured in a relief at the time of Queen Hatshepsut, fifteen centuries before Christ. But in spite of the fact that the ancient Egyptian has left us, in his beautiful stone carvings, a wonderful picture of the life of his day, the erection of obelisks was either so commonplace and simple in operation that it was never recorded, or records of this final work of the obelisk expert have been lost—or perhaps are still to be uncovered.

The Romans were the champion despoilers of the ancient world, and the movement of these huge blocks of stone from Egypt to Rome appears to have appealed to the Roman Emperors as a suitable means of demonstrating the



PULLEYS AND SNATCH BLOCKS

greatness and power of the Roman nation. Again, we know little of the means employed. In more recent times, however, the lowering, transportation, and reerection of obelisks has been considered no mean engineering feat, and the public press has devoted great space to describing such operations. Witness the erection of Cleopatra's Needle in New York and the equally famous obelisk task accomplished on the Thames Embankment in London.

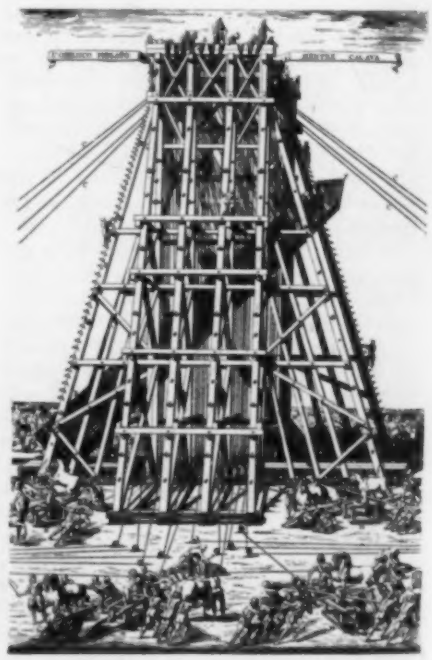
The story of how one of these monoliths was moved a few hundred feet and reerected in 1586, probably the first of modern attempts in this ancient art, is the subject of the following notes.

Early in the first century A.D., Emperor Caius Caligula brought from Heliopolis to Rome the shaft now known as the Vatican obelisk, which today stands in the center of the plaza in front of St. Peter's. Pliny describes it as having been erected by King Nuncoreus (probably Meneptah, 1322–1302 B.C.) in gratitude for the recovery of his sight. How this 325-ton shaft of granite, 88 ft long, was conveyed to Rome and set up does not appear to be recorded. It is known that Claudius erected it in the Circus of Nero, where so many early Christians were martyred. For fifteen centuries it remained there upright, the only one of the many brought to Rome to remain so. Even this one probably would ultimately have met the fate of its companions, for in 1585 Fontana found it in a dirty and unfrequented part of the city, tipped 17 in. from the vertical, its pedestal buried in rubbish and mud.

The problem of moving the shaft to a more prominent place in the city occupied the attention of several of the popes, but reviews of the project by the learned men of the day did not produce workable plans for accomplishing it until Sixtus V offered a prize for the best plan. On September 18, 1585, more than five hundred persons attended a meeting at which contestants presented their drawings, plans, and models. Among them was Dominicus Fontana (1543–1607) of Mili, on Lake Como, who planned to lower the shaft to a horizontal position, move it the required 825 ft on rollers, and reerect it with pulleys, ropes, and capstans. He illustrated his method to the assembly by actually manipulating a lead model of the obelisk with a miniature hoisting apparatus of strings and wood. His was declared to be the best plan and he was awarded the prize.

Since Fontana was only 42, he was thought to be too young to have charge of so huge an undertaking, and two older architects, Ammanati and Jacques de la Porte, who had gained repute in the moving of heavy weights, were selected to superintend the work. Fontana was not

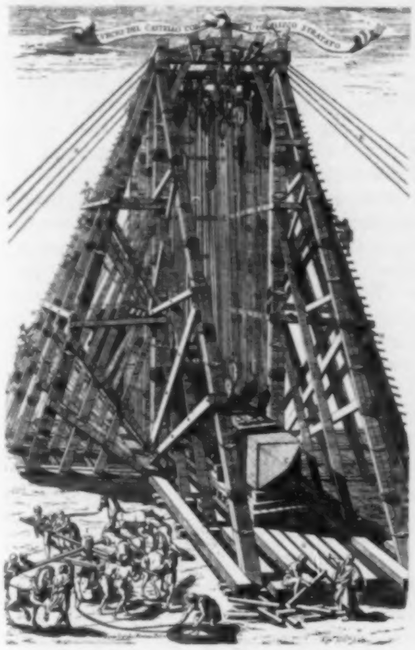
EFFECTING THE INITIAL LIFT



LOWERING ON TO THE CRADLE

pleased with this lack of confidence in his ability and after the work had begun let it be known that he was leaving Rome. When questioned by the Pope he said:

"At present but one idea fills my mind and absorbs my intellectual faculties. I am afraid modifications will be introduced into my system that may cause serious accidents for which I would be held partly responsible. The more I think of it the more convinced I am that injustice has been done me, for no one can carry out a design as well as the designer."



Ready to Move

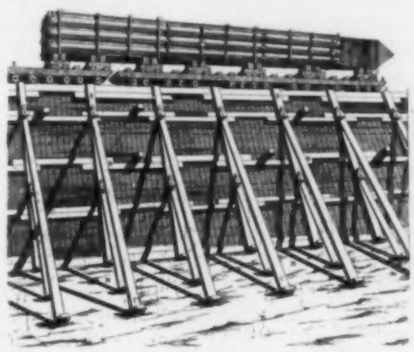
This plea won his case, and he was given full charge, with authority to take materials from anywhere in the Holy See and to demolish all buildings which interfered with the work.

This was a task of great magnitude. The story of its accomplishment is one of the most interesting chapters in the engineering history of the time. It attracted world attention; distinguished literary and scientific men from distant lands came to Rome to view the spectacle. It is fortunate that Fontana left a record of this achievement in a monumental book, *Templum Vaticanum*, published in Rome in 1694, which is replete with fine engravings by Carolus Fontana. From this volume the line cut on the Page of Special Interest and the smaller cuts accompanying these comments have been reproduced.

Armed with the full powers given him by Sixtus V, Fontana dispatched trusted agents to collect the timbers and equipment needed. From Campo Marto came the oak and walnut timbers that made up the scaffolding or "castle," 90 ft high, erected over the obelisk for the attachment of the lifting tackle. The posts of the "castle" were 40 in. square and were made up of four timbers each, carefully held together with iron bands and rope lashings. Only key bolts were used, so that after the tower was employed to lower the obelisk at the old location, it could easily be dismantled and reerected over the new site to raise the obelisk to its final position.

Fontana personally superintended the manufacture of the hemp ropes and the capstans. By test he determined the ultimate strength of the ropes and the mechanical advantage of the capstans. He carefully calculated the weight of the shaft, of the sheathing, and of the metal attachments to be 350 tons, and determined upon 40 capstans and tackles operated by 800 men and 75 horses as the principal motive power to accomplish the initial lifting of the obelisk. These were supplemented by 5 great levers 40 ft long, which were introduced under the shaft and by wood and iron wedges driven between the bottom of the shaft and its base. He assigned 10 men and 2 horses to each capstan—an amount of power insufficient to break the rope or the tackle. Thus, when the men and horses on a capstan hove to their utmost, they could make no further progress until other groups not taking their full share of the load had wound the proper tension into their ropes.

While the equipment was being assembled and the tower erected, the ground was cleared of interfering buildings, and a fence was built to enclose the entire area of the operations. Finally, on April 28, 1586, everything was ready. Before daybreak on the twenty-ninth Fontana received the benediction of the Pope, and he and his assistants took communion and attended two masses. Every road leading to the site was thronged with people. A proclamation of the death sentence was pronounced on anyone who forced his way through the enclosure or who broke the silence.



En Route to the New Location

Fontana exhorted the workmen to do their best; he recalled the signals—a blast of a trumpet to start heaving on the capstans, a stroke of a bell to stop—he visited all parts of the enclosure to satisfy himself that the equipment was in order and that all were in readiness to do their appointed task. Then he took a conspicuous position on a tower built for the purpose, and speaking so that all could hear recalled the religious motives which prompted the moving of the obelisk: "Implore with me the help of God, the sovereign moving power; let us ask for His help, without which all our efforts must be in vain." Workmen, noblemen, citizens, strangers, and priests all fell on their knees. Engineering undertakings of today lack the theatrical setting that the Renaissance could not do without.

The trumpet blast sounded, and the 906 men and 75 horses began to move, while Rome held its breath. Carpenters protected with metal helmets against falling or flying fragments drove their wedges under the shaft to support it as it rose. There was a stroke of the bell and everything stopped while a broken iron band was replaced with rope lashings. Another heave raised the shaft off the bronze crabs under the four corners, which had supported the shaft for fifteen centuries, supporting it at the corners of the base. At four in the afternoon the firing of a cannon announced that the obelisk had been lifted 2 ft, enough to insert the cradle beneath it. Amid rejoicing the batteries of the city responded with a salvo of salutes.

On the following day, while the obelisk rested on the wedges, steadied by the tackles, the bronze crabs were removed. Two of them, weighing 600 lb apiece, were leaded into the pedestal with long dovetailed spurs, so that four days and nights of chipping on the stone were required to remove them.

By the seventh of May preparations for the lowering had been completed. With struts of successively shorter length under the shaft to support its weight in case of failure of any of the tackles, the capstans were slowly slackened off. By four o'clock the obelisk was successfully landed on the cradle prepared for it. Fontana was escorted home with drums and trumpets amid shouts of universal greeting.

On succeeding days the cradle was moved to clear the pedestal, which was dug up and transported to its new location. In the bottom of an excavation 43 ft square and 24 ft deep, 9 in. chestnut and oak piles 18 ft long were driven in a solid

Ready to Lift to New Foundation

mass to form the support for an immense bed of puzzolana concrete, which nearly reached the ground surface. On this foundation the ancient pedestal was placed.

A timber ramp was built to this site, sloping to the elevation of the new pedestal; the timber scaffolding was re-erected over it; and the obelisk was slowly dragged by tackle and capstans along the ramp. By September 10, 1586, all these preliminaries had been completed, and the capstans and tackles were in position to lift the obelisk on to its pedestal. The same religious ceremonies were observed as before, and under the action of the 40 capstans worked by 140 horses and 800 men the monolith was slowly raised. By sunset, after 13 hours of heaving and twisting on the capstans, it had reached a vertical position and was separated from the pedestal only by the cradle.

Again, by means of the five levers, the wedges, and the 40 capstans worked in unison, it was lifted to free the cradle. The bronze crabs were replaced exactly as they had been found on the old pedestal, and after eight days of careful manipulation the wedges were withdrawn, the tackles were slackened off, and the obelisk was landed in its permanent position. The cost of its removal and reerection is recorded as $44,000.

It is said that during the operations an event occurred which Fontana did not record. It is a charming legend and may be true. At one time the success of the work was threatened by the stretching of a rope. A sailor, named Bresca, regardless of the death penalty shouted "Wet the rope" and thus saved the day. The legend goes on to say that Bresca was not only pardoned for his violation of the strict rule of silence but that the Pope gave him and his family forever a monopoly in the supplying of palm leaves for St. Peter's on Palm Sunday.

Rewards were heaped on Fontana for his achievement. He was made the pontifical architect and a Knight of the Golden Spur by Sixtus V; he received a pension, a gift of $6,000, and all the wood and material remaining from the undertaking, valued at $24,000. Three other obelisks in Rome were afterwards replaced by him. A fresco on the walls of the Vatican Library depicts the feat which raised this young engineer to sudden prominence.

Four years later Sixtus died and soon after, through the efforts of jealous enemies, Fontana was removed from his post. However, the Viceroy of Naples came to his rescue and made him architect and first engineer of the Kingdom of Naples, where he lived for many years, loaded with riches and honors. He designed many handsome buildings for the Viceroy.

Shortly after 1600 Fontana returned to Rome, this time in connection with the completion of the construction of St. Peter's. Michael Angelo had died without completing the huge dome, which was to be the crowning glory of his work. Fontana's reputation apparently led to his being called to finish this task, which he accomplished in a period of 22 months.

The narrative here given is taken largely from a chapter of the book, *Egyptian Obelisks*, by Henry H. Gorringe, New York, 1882. The chapter, "Reerection of the Vatican Obelisk," was prepared by Lt. Seaton Schroeder, U. S. Navy, who was the principal assistant to Commander Gorringe in the transportation of Cleopatra's Needle from Alexandria, Egypt, and its reerection in Central Park, New York, N.Y.

NEWS OF ENGINEERS

From Correspondence and Society Files

ROBERT M. JOHNSON is now chief of party for the U. S. Coast and Geodetic Survey on a CWA project of making local control surveys. His headquarters are in Sharptown, Md.

HAROLD M. PEARSON is at present employed as an inspector for the Civil Works Administration of Alameda County, California.

EDWARD D. MEYER has resigned as superintendent of construction for the Juul Construction Company, of Hutchinson, Minn., to become division engineer on the construction of the Transpersian Railway. He is employed by Consortium Kampsax, with headquarters in Teheran.

F. C. WHITNEY has accepted an engineering connection with the Bridge and Building Department of the City of St. Louis. He was formerly assistant engineer for William B. Ittner, of the same city.

GEORGE S. FROST has accepted an engineering position in the Pennsylvania Department of Health, with headquarters in Philadelphia. Previously he was senior assistant engineer in the Department of City (Philadelphia) Transit.

SIGMUND COHEN is now employed under the CWA as structural engineer in the City Engineer's Office, Department of Grade Separations, Detroit, Mich.

FREDERICK G. HEYDT, who was formerly with the M. F. C. Shoring and Foundation Company, of New York, N.Y., has now joined the staff of the Heydt-Mugler Company, Inc., shoring and scaffolding engineers, of the same city.

CHAD F. CALHOUN, who recently returned from the interior of Panama, where he was engineer for a mining expedition, has resumed his former position as engineer for the Henry J. Kaiser Company, of Oakland, Calif.

HARVEY A. SCHEEL, formerly an architectural draftsman with the Superintendent of Buildings and Grounds of the University of Chicago, is now chief of the alteration and space section of the Buildings and Grounds Division of a Century of Progress. His office is in the Administration Building, Burnham Park, Chicago.

FRANKLIN B. MARQUETTE has been made senior engineer of the Philadelphia City Planning Commission, in charge of transportation and highway studies, a CWA project.

PAUL C. SHAFER has resigned as city engineer of Ravenna, Ohio, to become superintendent of the Maysville Water Company, with headquarters in Maysville, Ky.

W. G. STROMQUIST, who was formerly director of the Bureau of Sanitary engineering and Division of Rural Sanitation of the Jefferson County (Alabama) Board of Health, has recently accepted an appointment as sanitary engineer for the Tennessee Valley Authority. His office is in Knoxville, Tenn.

ROY F. BESSEY is now regional planning consultant for the Federal Emergency Administration of Public Works, with headquarters in Portland, Ore.

SAMUEL T. CARPENTER has taken a position as assistant in the department of civil engineering of Ohio State University, at Columbus, Ohio. He was formerly a structural engineering apprentice with McClintic-Marshall, of Pittsburgh, Pa.

ROBERT P. BOYD has severed his connection with the Louisiana State Highway Commission to become senior highway design engineer with the U. S. Bureau of Public Roads in the district office at Fort Worth, Tex.

JOHN W. GORDANIER recently resigned his position with the U. S. Bureau of Reclamation to join the staff of the Six Companies Inc., on the construction of Boulder Dam. His headquarters are Boulder City, Nev.

GLOSTER P. HEVENOR, formerly secretary-treasurer of Hevenor and Weller, Inc., of Rochester, N.Y., has been appointed vice-president and general manager of the Johnson-March Corporation, of New York, N.Y.

G. A. SEDGWICK has joined the staff of the California State Highway Commission in the capacity of assistant bridge designing engineer. His headquarters are in Sacramento, Calif.

WILLIAM J. BALDWIN, JR., has been promoted from the position of assistant secretary and chief engineer of the New York Steam Corporation, of New York, N.Y., to that of secretary-manager of commercial relations of this organization.

M. JUUL HVORSLEV is at present engaged in research work in Professor Terzaghi's laboratory for soil mechanics, in Vienna, where he will remain until the summer of 1934.

M. E. Salsbury has resigned as an instructor in hydraulics at the University of Southern California, to become an investigation engineer for the Los Angeles County Flood Control District, with headquarters in Los Angeles, Calif.

Jackson H. Wilkinson, formerly structural and hydraulic designer for the Morgan Engineering Company, of Bethany, Ill., has joined the staff of the Tennessee Valley Authority, in Knoxville, Tenn.

Elmer J. Christenson has been promoted from the position of junior engineer in the U. S. Engineer Office at Alma, Wis., to that of assistant engineer in the same office.

Eugene J. Peltier has taken a position as instrumentman with the Kansas State Highway Commission. His headquarters are Norton, Kans.

Walter L. Webb has recently been appointed chief mathematician for the State of Pennsylvania in the U. S. Coast and Geodetic Survey.

Jack E. DeMuth is now connected with the Austin Western Road Machinery Company, with headquarters in Omaha, Nebr. He was formerly in the employ of the Western Wheeled Scraper Company, of Aurora, Ill.

Michael E. Milone, formerly chief of party for Carl B. Lovell, of New York, N.Y., has now become an assistant engineer with the Board of Transportation of the same city.

R. C. Hicks has joined the staff of the Michigan State Highway Department in the capacity of project engineer. His headquarters are Albion, Mich.

L. M. Shumaker, who was formerly employed by the Stoecker Engineering Company, of St. Louis, Mo., is now sewer designer for that city.

William H. Gravell is now state engineer of the Public Works Administration of Pennsylvania, with offices in Harrisburg, Pa.

Samuel H. Lea has been appointed an engineer-examiner for the Public Works Administration, with headquarters in Washington, D.C.

F. R. Michelsen has recently accepted a position with Consortium Kampsax on the construction of the Transpersian Railroad, with offices in Teheran, Persia.

Lewis J. H. Grossart has taken the position of town engineer of Catasauqua and Freemansburg, Pa., and of Civil Works Administrator for Lehigh County. His headquarters are Allentown, Pa.

William R. Johnston is now superintendent of construction of the Veterans' Administration Facility, with offices in Roanoke, Va.

Thad A. Kay has been promoted from the position of designer for the Freyn Engineering Company, of Chicago, Ill., to that of principal structural engineer of the same organization.

Harold W. Hudson has resigned as Assistant State Highway Engineer of New Jersey to accept a position as engineer of construction of the Triborough Bridge Authority, with headquarters in New York, N.Y.

Allan S. Beale is now resident engineer for Fay, Spofford and Thorndike, consulting engineers, Bourne, Mass.

E. T. Roetman, formerly research fellow in sanitary engineering at the University of West Virginia, has taken the position of engineer for the Tygart Valley Homesteads, Inc., with offices in Elkins, W. Va.

O. H. Ammann, chief engineer of the Port of New York Authority, has been loaned by the Port Authority to serve as chief engineer of the Triborough Bridge Authority.

James W. Porter has accepted an engineering position with the Gifford-Hill Pipe Company, of Dallas, Tex.

Darwin W. Townsend announces that he has entered private consulting engineering practice as a member of the firm of Consoer, Older and Quinlan, of Chicago, Ill., with offices to be located in Milwaukee, Wis. He has resigned as acting chief engineer of the Milwaukee and Metropolitan sewerage commissions.

Charles A. Case has accepted the position of district maintenance engineer of the Illinois State Highway Department, with headquarters in Chicago, Ill. Previously he was field engineer for the Cook County Highway Department.

Cornelius C. Vermeule, Jr., is now state engineer of the Public Works Administration of New Jersey, with headquarters in Newark, N.J.

Robert L. Hahn has taken a position as division engineer with the State Highway Commission of Kansas, with headquarters in Garden City, Kans.

Archer W. Bedell, formerly city engineer of Faribault, Minn., has joined the staff of the Minnesota State Highway Department in the capacity of right-of-way field agent. His office is in St. Paul, Minn.

George J. Davis, Jr., is now state engineer of the Public Works Administration of Alabama, with headquarters in Montgomery, Ala. He was formerly professor of civil engineering at the University of Alabama.

Henry Weinstein recently received an appointment as a senior draftsman in the U. S. Engineer Office at Glasgow, Mont.

John Latenser, Jr., has accepted an appointment as state engineer for the Public Works Administration of Nebraska, with offices in Omaha, Nebr.

Harold B. Pullar has resigned as secretary of the James B. Berry's Sons Company, of Chicago, Ill., to accept a connection with the American Mexican Petroleum Corporation, of the same city.

Mortimer E. Cooley, dean emeritus of the colleges of engineering and architecture of the University of Michigan, is now serving as state engineer of the Public Works Administration of Michigan, with headquarters in Detroit.

Herman Schorer, formerly an engineer with the U. S. Bureau of Reclamation, in Denver, Colo., has joined the staff of Borsari and Company, of New York, N.Y.

Charles W. Palmer has taken a position as assistant engineer in the Design Division of the Department of City Transit of Philadelphia, Pa.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From February 10 to March 9, 1933, Inclusive

Additions to Membership

Baihly, William Edington (Jun. '34), 814 South Sheridan, Tacoma, Wash.

Bardin, William Joseph (Jun. '34), 460 North 1st St., San Jose, Calif.

Barrett, Eugene Vincent (Assoc. M. '34), Testing Engr., Civ. Eng. Testing Laboratory, Columbia Univ., New York (Res. 209 Westmoreland Ave., White Plains), N.Y.

Boley, Arthur Leland (M. '33), City Engr., City Hall, Sheboygan, Wis.

Bollman, Leslie Weber (Jun. '34), Transitman, S. J. Reid, Inc., Flushing (Res., 22–58 Forty-eighth St., Long Island City), N.Y.

Bringhurst, John Henry, Jr., (Jun. '34), Asst. Civ. Engr., Gulf Production Co., 1805 Gulf Bldg. (Res., 2609 Crocker St.), Houston, Tex.

Chayabongse, Chamras (Jun. '34), Care, The Siamese Legation, 2300 Kalorama Rd., Washington, D.C.

Corsan, Benjamin (Jun. '33), 1233 Evergreen Ave., New York, N.Y.

Cox, John Luther (Jun. '32), With Los Angeles County Flood Control Dist.; 615 North Olive Ave., Alhambra, Calif.

DeCerce, Frank (Jun. '33), 950 Emerson Ave., Syracuse, N.Y.

DEVOE, STEPHEN JOSEPH, JR., (Jun. '33), 531 East Lincoln Ave., Apartment 7 P, Mount Vernon, N.Y.

DOWNS, LEONARD VAUGHN (Jun. '34), Junior Engr., U.S. Bureau of Reclamation, 226 U.S. Custom House, Denver, Colo.

DUNBAR, T. J., JR., (Jun. '33), 2504 Rio Grande, Austin, Tex.

EVANS, JACK CLARKSON (Jun '34), 471 Grace Ave., West Plains, Mo.

FARNEY, HAROLD SAMUEL (Jun. '34), Castorland, N.Y.

FEICHTMEIR, ARMAND CASIMIR (Jun. '33), Box 330, Eureka, Calif.

FERBRACHE, CHARLES PHILIP (Jun. '33), 1100 Main St., Rolla, Mo.

FOSTER, WENDELL LEE (Jun. '33), County Engr., Woods County, County Engrs. Office, Alva, Okla.

FRAAD, HENRY OBER (Assoc. M. '34), Chf. Engr., Allied Pneumatic Services, Inc., 347 Fifth Ave., New York, N.Y.

FREDERICK, HARRY ARTHUR (Jun. '33), Laboratory Asst., Public Elec. & Gas Co., 938 Clinton Ave., Irvington (Res., 399 Lincoln Ave., Orange), N.J.

FULLER, WILLIAM JOHN (M. '34), Prof., Structural Eng., Univ. of Wisconsin, 623 West State St., Milwaukee, Wis.

HEFLIN, CARL WASHINGTON (Assoc. M. '34), Field Engr. Constr. Dept., B. & O. R.R., Stapleton (Res., 71 Central Ave., St. George), N.Y.

HENRY, ARTHUR RICHMOND (Jun. '34), Road Supervisor, Borough of Vineland; 132 Brentwood Ave., Pitman, N.J.

HENSHAW, LAMOND FORBES (Jun. '34), Junior Engr., War Dept., Corps of Engrs., 341 Pittock Blk., Portland, Ore.

JOHNSON, ALBERT EDWIN (Jun. '33), 360 North Capitol St., Salem, Ore.

JOY, DONALD GORDON (Jun. '33), 86-35 Homer Lee Ave., Jamaica Estates, N.Y.

KJERULF, HANS FREDRIK (Assoc. M. '34), With Design and Estimating Dept., McClintic-Marshall Corporation (Res., 111 East Raspberry St.), Bethlehem, Pa.

LENHART, JACK (Jun. '33), 3939 Quincy St., N.E., Minneapolis, Minn.

LUDEMAN, RICHARD HOUSTON (Jun. '33), 25 Clark St., Apartment 314, Brooklyn, N.Y.

MCLAUGHLIN, CHILTON WHITE, JR. (Jun. '34), County Surv., Wyandotte County, Court House, Kansas City, Kans.

MARTA, GEORGE ARTHUR (Jun. '33), 943 Jackson St., Philadelphia, Pa.

MATHOFF, DAVID (Assoc. M. '34), 69 Myrtle St., Boston, Mass.

MILLARD, LESTER WARD (Assoc. M. '34), Acting Bridge Engr., State Highway Dept. (Res., 515 West St.), Lansing, Mich.

MONTEALEGRE, FRANCISCO GERMAN, JR., (Assoc. M. '34), 3585 Clay St., San Francisco, Calif.

MORRIS, SETH BRADLEY (Assoc. M. '34), 106 South Gregory St., Urbana, Ill.

NUDENBERG, NATHANIEL JOHN (Jun. '33), 26 Leo Pl., Newark, N.J.

NUTE, JOHN WARREN (Jun. '33), Box 446, Los Altos, Calif.

OMAN, ARLIE ANDERS (Jun. '33), 406 Fifth St., Bismarck, N. Dak.

ORIVE ALBA, ADOLFO (Jun. '34), Chf. Engr., Mexican Irrig. Comm., Patzcuaro 185, Chapultepec Heights, City of Mexico, Mexico.

ORR, GERRY MITCHELL (Jun. '34), Box 152, Fordyce, Ark.

PHILLIPS, NORMAN, JR., (Jun. '33), Blacksburg, Va.

PIKE, THOMAS OLIVER (Jun. '33), Pump Barge No. 3, Dredging Area, Natchez, Miss.

PUGH, CLIFFORD ARTHUR (Jun. '33), 2380 Atlantic Ave., Long Beach, Calif.

REISNER, ROBERT HENRY (Jun. '33), Care, State Highway Dept., Salem, Ore.

RICKENBERG, EDWARD HENRY (Jun. '33), 1071 Dodd St., Napoleon, Ohio.

ROGERS, FRANKLIN (M. '32), Traveling Representative for Dist. Engr., 8th Dist., Civ. Works Administration, 511 South Neil St., Champaign, Ill.

SCHOLTZ, WALTER (Jun. '34), Asst. Shop Foreman, New England Pacific Screw Metal Products Co. (Res., 1929 North Hobart Boulevard), Los Angeles, Calif.

SETIAN, HAIG (Jun. '33), 308 North Front St., Philadelphia, Pa.

SPORSEEN, STANLEY EMANUEL (Jun. '33), Care, Camp Union, Bremerton, Wash.

SUTER, MAX (M. '34), 401 North Race St., Urbana, Ill.

SWANSON, CARL GUSTAVE WALTER (Jun. '33), 944 North East 2d Ave., Miami, Fla.

TEFFT, ROBERT HOWARD (M. '34), Chf. Engr., Cia. Agricola Carabayllo Cartaria of Peru, 166 Walnut St., Nutley, N.J.

TEN EYCK, PETER GANSEVOORT (M. '34), Chairman, Albany Port Dist. Comm., 74 Chapel St., Albany, N.Y.

TILLAPAUGH, HOWARD WESLEY (Jun. '33), Box 247, Charles St., Torrington, Conn.

TRAVIS, WAYNE IVAN (Jun. '34), Junior Engr., U.S. Geological Survey, Federal Bldg., Boise, Idaho.

VALENSTEIN, SAMUEL (M. '34), Chf. Engr., M. Shapiro & Son, 1560 Broadway, New York, N.Y.

WHITE, EDWARD EMELIN (Jun. '34), Parkview Apartments, Winona, Minn.

WOODS, GEORGE ALAN (Jun. '33), 5621 Sylvan Ave., New York, N.Y.

WYLY, LAWRENCE THEODORE (M. '34), Res. Engr., State Div. of Highways, Ottawa (Res., 8633 Vernon Ave., Chicago), Ill.

MEMBERSHIP TRANSFERS

CRAGGS, CLYDE (Assoc. M. '27; M. '34), Chf. Engr., Worden-Allen Co. and Permanent Constr. Co. (Res., 5718 Winthrop Ave.), Chicago, Ill.

HAINES, ELTON LEE (Jun. '29; Assoc. M. '34), Engr., The R. Hardesty Mfg. Co., Box 2170, Denver, Colo.

HENNY, ARNOLD LORENTZ (Jun. '30; Assoc. M. '34), Care, U.S. Bureau of Reclamation, 440 Custom House, Denver, Colo.

KAY, CARNIG (Jun. '29; Assoc. M. '34), Asst. Engr., Springfield Water Works, Cobble Mountain Reservoir, Blandford, Mass.

LORENZINI, ERNEST MAURICE (Jun. '30; Assoc. M. '34), Engr., Am. Bitumuls Co., 229 Broadway, Everett (Res., 21 Cross St., Belmont), Mass.

LYLE, ALEXANDER (Assoc. M. '23; M. '33), 66 Vinton St., Long Beach, N.Y.

MEAD, WILLIAM HENRY (Assoc. M. '17; M. '34), Chf. Engr. and Gen. Supt., Salt Flat Water Co. and Darst Salt Water Co., Box 733, Luling, Tex.

MILLER, GEORGE HODGSON (Jun. '27; Assoc. M. '34), Engr. Examiner with State Engr. on Federal Emergency Administration of Public Works, Box 84, Moscow, Idaho.

MOORE, LEWIS BLAFFER (Assoc. M. '28; M. '34), Asst. Office Engr., Panama Canal, Box 246, Balboa Heights, Canal Zone.

ORTOLANI, WALTER ALBERT (Jun. '26; Assoc. M., '34), Res. Engr. for Wharton County, State Highway Dept., Box 73, Wharton, Tex.

PORTER, EARL LEONARD FRANKLIN (Jun. '31; Assoc. M. '34), Junior Engr., U.S. Engr. Field Office, Alma, Wis.

SCHWERIN, BENJAMIN (Assoc. M. '26; M. '33), Asst. Engr., Dept. of Public Works of Manhattan, Municipal Bldg. (Res., 2205 Ryer Ave.), New York, N.Y.

TOTAL MEMBERSHIP AS OF MARCH 9, 1934	
Members	5,759
Associate Members	6,261
Corporate Members	12,020
Honorary Members	18
Juniors	3,105
Affiliates	108
Fellows	4
Total	15,255

SKYTTE, JOHANNES (Assoc. M. '26; M. '34), Hydr. Eng. Designer, City of San Francisco, Hetch Hetchy Dept., 425 Mason St., San Francisco, Calif.

STRAUB, LORENZ GEORGE (Jun. '28; Assoc. M. '34), Associate Prof. of Hydraulics and Head, Hydraulics Div., Univ. of Minnesota, Experimental Eng. Laboratories, Univ. of Minnesota, Minneapolis, Minn.

TCHIKOFF, VALENTINE VASILIEVISH (Assoc. M. '19; M. '34), 317 West 99th St., New York, N.Y.

THEIMER, OTTO FREDERICK (Assoc. M. '26; M. '34), Cons. Engr., Ausstellungsstrasse 12a, Brno, Czechoslovakia.

TOMLINSON, GEORGE EDMUND (Jun. '27; Assoc. M. '34), Supervisor, Eng. Training, Tennessee Val. Authority, Knoxville, Tenn.

TSENG, TSAO WEN (Jun. '30; Assoc. M. '33), Section Engr., Hua-Ying-Hua-Chow Section, Lunghai Ry.; 3 East Small Bridge, Soochow, Ku, China.

TUDOR, RALPH ARNOLD (Jun. '30; Assoc. M. '34), Senior Designing Engr. of Bridges, Div. of Highways, State Dept. of Public Works, San Francisco-Oakland Bay Bridge, 500 Sansome St., San Francisco (Res., 1016 Harvard Rd., Oakland), Calif.

WOLFF, WILLIAM ROBERT (Jun. '29; Assoc. M. '34), Water Engr., Public Service Comm., 80 Centre St., New York, N.Y.

REINSTATEMENTS

DEAN, WILLIAM ENNELS, JR., Jun., reinstated Feb. 19, 1934.

KALES, WILLIAM ROBERT, M., reinstated Feb. 19, 1934.

LEESON, ROBERT VONARTIEVELDE, M., reinstated Feb. 15, 1934.

RESIGNATIONS

DUNGLINSON, GEORGE, JR., Assoc. M., resigned Feb. 7, 1934.

KRUG, ARTHUR GILLUM, M., resigned Feb. 15, 1934.

MCGUINNESS, WILLIAM JAMES, Jun., resigned Feb. 26, 1934.

OSBORNE, SIDNEY HOWARD, M., resigned Feb. 28, 1934.

ZIPSER, MORRIS ERNEST, Assoc. M., resigned Feb. 13, 1934.

DEATHS

BARKER, HAROLD WARD. Elected Assoc. M., April 3, 1922; died Feb. 14, 1934.

EATON, ARTHUR CHESTER. Elected M., July 6, 1920; died March 6, 1934.

FLINT, CHARLES RANLETT, Fellow, June 7, 1876; died Feb. 12, 1934.

GIBBS, ELBERT ALLAN. Elected Jun., March 6, 1906; Assoc. M., July 1, 1909; died March 6, 1934.

HULL, WILLIAM HENRY. Elected M., May 19, 1924; died Sept., 26, 1933.

JOHNSON, JAMES MORELAND. Elected M., July 1, 1891; died Feb. 11, 1934.

KERNOT, MAURICE EDWIN. Elected M., June 5, 1907; died Jan. 13, 1934.

LYON, FREDERIC WILLIAM. Elected Jun., Oct. 3, 1911; Assoc. M., Dec. 31, 1913; M., Oct. 8, 1918; died Nov. 11, 1933.

MACBEATH, DAVID LIVINGSTONE. Elected Assoc. M., Aug. 28, 1922; died Jan. 26, 1934.

PARMLEY, WALTER CAMP. Elected Assoc. M., April 1, 1896; M., June 1, 1898; died Feb. 19, 1934.

SCHLICH, OTTO STEPHEN. Elected Assoc. M., Jan. 16, 1922; M., Dec. 15, 1924; died December 1933.

SCHUTT, HERBERT DAVIS. Elected Assoc. M., April 23, 1928; date of death unknown.

SHIPMAN, EUGENE HICKS. Elected M., May 2, 1911; died Feb. 14, 1934.

STAEHLE, GILBERT COBB. Elected Jun., Dec. 3, 1913; Assoc. M., Nov. 27, 1917; died Feb. 6, 1934.

VINCENT, EDWIN DERICKSON. Elected M., May 4, 1909; died March 4, 1934.